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Joint Dry/Wet Weather Treatment of Municipal Wastewater at Clatskanie, Oregon

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Portland, OR

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JOINT DRY/WET WEATHER TREATMENT OF MUNICIPAL WASTEWATER AT CLATSKANIE, OREGON

by

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Grant No. R-806003

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development comprise that necessary first step in problem solution and involve defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and for the minimization of adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of such research; a most vital communications link between the researcher and the user community.

This study describes the two year evaluation of joint dry/wet weather treatment of Municipal Wastewater at Clatskanie, Oregon. The wet weather treatment capacity reduces pollution discharges caused by excessive infiltration/inflows.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

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ABSTRACT

This report describes the two year plant scale evaluation of physical and biological processes used for joint treatment of dry weather and storm generated sanitary sewer flow. The project was conducted in Clatskanie, Oregon at the City's new joint dry/wet weather sewage treatment plant. The purpose of the project was to research and define the performance capability of the new plant for potential nation-wide application. As part of the project, economic and technical criteria for developing alternative I/I control methodologies were reviewed and are presented as part of the report.

The plant uses primary clarification with activated sludge secondary, the present dry weather flow (DWF) is 0.19 mgd (719 m3/day) and the ultimate DWF capacity is 0.5 mgd (1892 m3/day). The secondary process has the capability of being run in a contact-stabilization mode, and the primary clarifier can be operated in a dissolved air flotation (DAF) mode. With these process modifications, flows up to 1.25 mgd (4731 m3/day) or more than 6 times present DWF can be effectively treated.

Results of the evaluation program showed that based on mean values, the DAF-contact stabilization treatment system configuration maintained effluent TSS and BOD concentrations of 2-24 mg/l and 6-11 mg/l, respectively, for average flows up to 1.07 mgd (4050 m3/day) or 6 times DWF. During DWF plant removal efficiencies for TSS and BOD were 94%. Wet weather flow (WWF) removal efficiencies for storms with peak flows ranging from 0.53 mgd to 2.31 mgd (1892 to 8743 m3/day) or 3 to 12 times DWF were 71% and 73% for TSS and BOD respectively.

The capital cost of the DAF-contact stabilization capability was estimated to be 14% over the cost of a standard DWF plant. The total cost, including O & M, was estimated to be 10% over the cost of a standard DWF plant. The unit treatment cost for the total volume treated was determined to be $1.45/1000 gal ($0.38/m3) and $1.33/1000 gal ($0.35/m3) for just the storm generated segment of flow. A cost effective analysis comparison of joint dry/wet treatment, sewer system rehabilitation and flow equalization storage for the City of Clatskanie showed that joint treatment was considerably less expensive and was nearly as effective in controlling I/I discharges.

This report was submitted in fulfillment of Grant No. R-806003 under the sponsorship of the U.S. Environmental Protection Agency and the City of Clatskanie, Oregon. The evaluation program was performed by Whiteley-Jacobsen and Associates acting as a subcontractor to the grantee, the City of Clatskanie. Work was completed in September of 1980.

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SECTION 1

INTRODUCTION

Techniques oriented toward the management and control of dry weather sewage flow (DWF) have been widely studied and applied in the United States. However, it has only been in the last decade or so that nation-wide, clean water objectives have been broadened to specifically recognize contaminated wet weather flow (WWF) as a major source of water pollution.

Whether in the form of storm water drainage from sewered and unsewered collection areas, or as infiltration/inflow (I/I) combined with municipal wastes, these discharges have been shown to contribute substantial loads of organics, solids, nutrients and other pollutants to receiving waters (1). Such loads have caused direct impairment of water quality, as well as impacts to the indigenous aquatic ecosystem.

Potential control measures for wet weather discharges generally fall into four broad categories: Land management alternatives, collection system controls, storage and release techniques, and direct end-of-pipe treatment. In many cases site-specific conditions require the application of one or more of these measures to successfully alter quantity (flow) and/or quality of excess water discharged.

This report describes the results of a research and demonstration study on a full-scale treatment facility processing wastewaters for the City of Clatskanie, Oregon. The facility has been designed to treat a wide range of flows that are generated during both dry and wet weather periods.

Basis for Study

The City of Clatskanie, Oregon is located in rural northwest Oregon, within 3 miles (4.8 km) of the Columbia River, and approximately 50 miles (80 km) north-northwest of the Portland metropolitan region. The City encompasses an area of 650 Acres (263 Hectare) and has a population of 1750.

Approximately 85 percent of the City's population is presently sewered, accounting for roughly 70 percent of the City's geographical area. Loads to the sewer system are primarily domestic as no major industrial discharges are connected to the system.

In 1971 the Oregon State Department of Environmental Quality conducted sampling on the effluent from Clatskanie's existing trickling filtration plant. Because of low BOD (5-day, Biochemical Oxygen Demand) and suspended solids (SS) removals, the Department requested the City to undertake modifications to improve performance. A preliminary investigation was also made that same year which showed that the collection system was subjected to severe I/I and that, in addition, deficiencies existed in the treatment plant's process capabilities.

In 1974 the City conducted an infiltration/inflow study under Public Law 92-500. The results of that study identified the extent of I/I in the system and proposed several alternatives for meeting the necessary pollution control objectives. These alternatives ranged from sewer rehabilitation with expansion of the existing trickling filter system to development of a new treatment facility designed to accommodate the full range of wet and dry weather flows expected.

Cost effectiveness analysis indicated that the latter alternative - development of a joint dry/wet weather facility without major collection system improvements - was the best solution at Clatskanie. Application of this alternative centered on the development of a treatment plant employing a single primary clarifier capable of dissolved air flotation (DAF) with high wet weather flows, followed by contact stabilization activated sludge having sufficient process flexibility to handle the entire range of flows anticipated. Design of the system was initiated in 1974 and construction was completed in January, 1978.

Research Study

The unique design concept represented by the construction of a single facility to treat both dry and wet weather flows at Clatskanie prompted the City to seek a grant from the U.S. Environmental Protection Agency (EPA) to research and define the performance capability of the new plant for potential nation-wide application. As part of the project, economic and technical criteria for developing alternative I/I control methodologies were reviewed and are presented as part of the report. The proposal for this study was approved and funds awarded on May 31, 1978, under EPA Grant No. R-806003. The study was conducted over a two-year period between July, 1978 and June, 1980.

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SECTION 2

CONCLUSIONS

1. Based on mean values obtained in this study, the DAF-Contact stabilization treatment system configuration used at Clatskanie is capable of maintaining effluent TSS and BOD concentrations of 2-24 mg/l and 6-11 mg/l, respectively, for average flows up to 1.07 mgd (4050 m3/day) or 5.6 times DWF. However, transient concentrations exceeding these values can be expected.

2. With the DAF-Contact Stabilization mode, wet weather flows up to 1.25 mgd (4731 m3/day) or 6.6 times DWF can be treated to achieve effluent TSS and BOD concentrations equivalent to those which are considered representative of dry weather flow activated sludge operation. Under the conditions stated, the effluent BOD concentration was less than 15 mg/l 98% of the time, and TSS concentration was less than 20 mg/l 80% of the time, with the flow below 0.84 mgd 70% of the time.

3. Two different flush phenomena were observed at Clatskanie. The storm to storm flush occurred first. It primarily consisted of light organic matter which was deposited between storms. This flush occurred at flows ranging from 0.4 to 0.6 mgd (1500 to 2300 m3/day), 2 to 3 times DWF. The second flush phenomenon was the seasonal flush. This flush phenomena occurred during the wet season as successively higher storm flows were achieved. The seasonal flush consisted mainly of heavier inorganics which were deposited in the sewer over the entire year. Fine inorganics associated with this flush proved to be difficult to remove.

4. The results of this study support a wet weather flow criterion of 2100 mg/l as a minimum contact basin mixed liquor suspended solids concentration.

5. In Contact Stabilization treatment TSS and BOD concentrations discharged under wet weather flows can be controlled with contact times as low as 13 minutes and reaeration times as low as 1.9 hours.

6. Plant performance during long term and short term storms may be characterized by the following samples:

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Long Term Storm -- The storm which occurred Janaury 11, 1980 lasted approximately six days. The flows ranged from 0.45 to 2.31 mgd (1703 to 8743 m3/day) with an average flow of 1.09 mgd (4126 m3/day). The maximum flow was 12 times DWF and the average flow was 6 times DWF. The average effluent BOD concentration and mass rate were 10 mg/l and 90 lb/day (41 kg/day) respectively. The average effluent TSS concentration and mass rate were 25 mg/l and 225 lb/day (102 kg/day) respectively. The BOD and TSS removal efficiencies were 72% and 61% respectively. The plant was hydraulically overloaded at the peak flow rate and storm to storm and seasonal flushes occurred. Removal efficiencies were depressed by dilute raw sewage. Minimum influent BOD and TSS concentrations were 18 mg/l and 28 mg/l respectively. Fine inorganics passing through the plant were partially responsible for the low TSS removal efficiency.

Short Term Storm -- The storm which occurred November 22, 1979 lasted approximately 13 hours. The flows ranged from 0.49 to 0.80 mgd (1855 to 3028 m3/day) and the average flow was 0.61 mgd (2309 m3/day). The maximum flow was 4 times DWF and the average flow was 3 times DWF. The average effluent BOD concentration and mass rate were 20 mg/l and 103 lb/day (47 kg/day) respectively. The average effluent TSS concentration and mass rate were 1.8 mg/l and 10 lb/day (5 kg/day) respectively. The BOD and TSS removal efficiencies were 84% and 99% respectively. A storm to storm flush occurred during this storm and was contained by the plant. The effluent TSS concentration was abnormally low, but the effluent BOD concentration was somewhat high. The high BOD concentration may have been the result of the contact MLSS concentration falling slightly below 2100 mg/l.

Overall removal efficiencies for both long term and short term storms were 73% and 71% for BOD and TSS respectively. These values are based on the total mass amounts for storms monitored in the 1979-1980 wet season.

7. For this installation, the DAF-Contact Stabilization capability required an additional 14% in capital cost, and an additional 10% in total cost.

8. When compared to sewer system rehabilitation and flow equalization storage at Clatskanie, joint dry/wet weather treatment, although it does not offer the most complete control of I/I, does offer the most I/I control per dollar spent. Annual mass removals of BOD and TSS were 93% for joint treatment as opposed to estimated 94% for rehabilitation and flow equalization. Joint treatment as an I/I control method cost less than half as much as either of the other methods.

9. Of the seven metals monitored during the study, three, chromium, cadimium and mercury were not present in detectable quantities. The four remaining, lead, copper, zinc and nickel, were present in raw

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waste concentrations ranging from 0.020 to 0.044 mg/l except for zinc which ranged from 0.14 to 0.26 mg/l. Generally, very little removal of the metals was accomplished by the primary process, but significant removal was accomplished in the secondary process for all metals except nickel.

10. Kjeldahl nitrogen raw waste concentration was 22 mg/l for DWF and 15.2 mg/l for WWF. Total phosphorus raw waste concentrations were 5.7 mg/l and 2.0 mg/l for DWF and WWF respectively. Generally, very little nutrient removal occurred in the primary process, but significant removal occurred in the secondary process. Overall plant removal of kjeldahl nitrogen averaged 67%, while overall plant removal of total phosphorus averaged 31%.

11. Dissolved air flotation is capable of achieving up to 66% TSS removal and up to 45% BOD removal.

12. Operationally, primary settled sludge which accumulated during dry weather flow should be withdrawn prior to use of dissolved air flotation otherwise removal of suspended solids will be affected.

13. Excessive skimmer speeds decrease removal of TSS during dissolved air flotation operation.

14. Deterioration of TSS and BOD removal by dissolved air flotation occurs at overflow rates, based on raw waste flow, in excess of 5000 gpd/ft2 (200 m3/m2/day).

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SECTION 3

RECOMMENDATIONS

It is recommended that:

1. The DAF-Contact Stabilization capability be considered for incorporation into treatment systems for cities with infiltration-inflow problems. The process modifications are comparatively inexpensive and secondary effluent standards can be maintained.

2. DAF clarification be evaluated as a final clarifier.

3. Future DAF-Contact Stabilization installations utilize either a DAF secondary clarifier, or a two clarifier scheme. This would avoid operational problems encountered treating dry weather flows with an oversized secondary clarifier.

4. Twenty four hour a day operation be provided at future DAF-Contact Stabilization facilities. If this is not practical, the mechanism should be provided for the secondary process to be flipped over to Contact Stabilization automatically, as plant flows increase.

5. Coagulants be tried in the DAF unit at Clatskanie to facilitate removal of fine inorganic particles.

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SECTION 4

DESCRIPTION OF STUDY SITE

The City of Clatskanie, Oregon is situated at the mouth of a narrow valley formed, in part, by the Clatskanie River and its tributaries. Although a large part of the City, including the central business district, lies on flat or gently sloping land, several outlying areas increase in elevation by over 100 feet (30m), rising at slopes greater than 30-40 percent.

A deposit of light brown, massive sandy and silty loess, in depths as great as 100 feet (30 m) mantles much of the lower area of the City. This homogeneous, structureless silt is composed predominately of quartz and other minerals resulting from the glacial outwash of the nearby Columbia River. The soils have moderately-developed, moderately-fine to fine textured profiles. Principal topographic and geographic features of the City and of the nearby areas are shown in Figure 1.

Sewage Collection Facilities

Clatskanie's existing sewer service area encompasses roughly 455 acres (184 hectare). A map of the sewer service area showing general features of the collection system is given in Figure 2.

The collection system in Clatskanie dates back to about 1910. At that time terra cotta sewer lines with concrete joints were installed in several streets located in the central business district. In the period between 1955 and 1957 the system experienced a major expansion using concrete and transite pipe with rubber and mortar joints. In 1965 the transite pipe system was extended to include a new subdivision west of the central business district. Additional lines have been connected to the system on a regular basis up to the present.

Pipe size used in the collection system varies from six to fifteen inches (15 to 38 cm) in diameter. There are 3 lift stations and one main pumping station in the system.

The service area consists of two principal drainage zones. For areas north of the Clatskanie River wastewater is collected and transmitted to lift station no. 1-A, where it is pumped to the treatment plant.

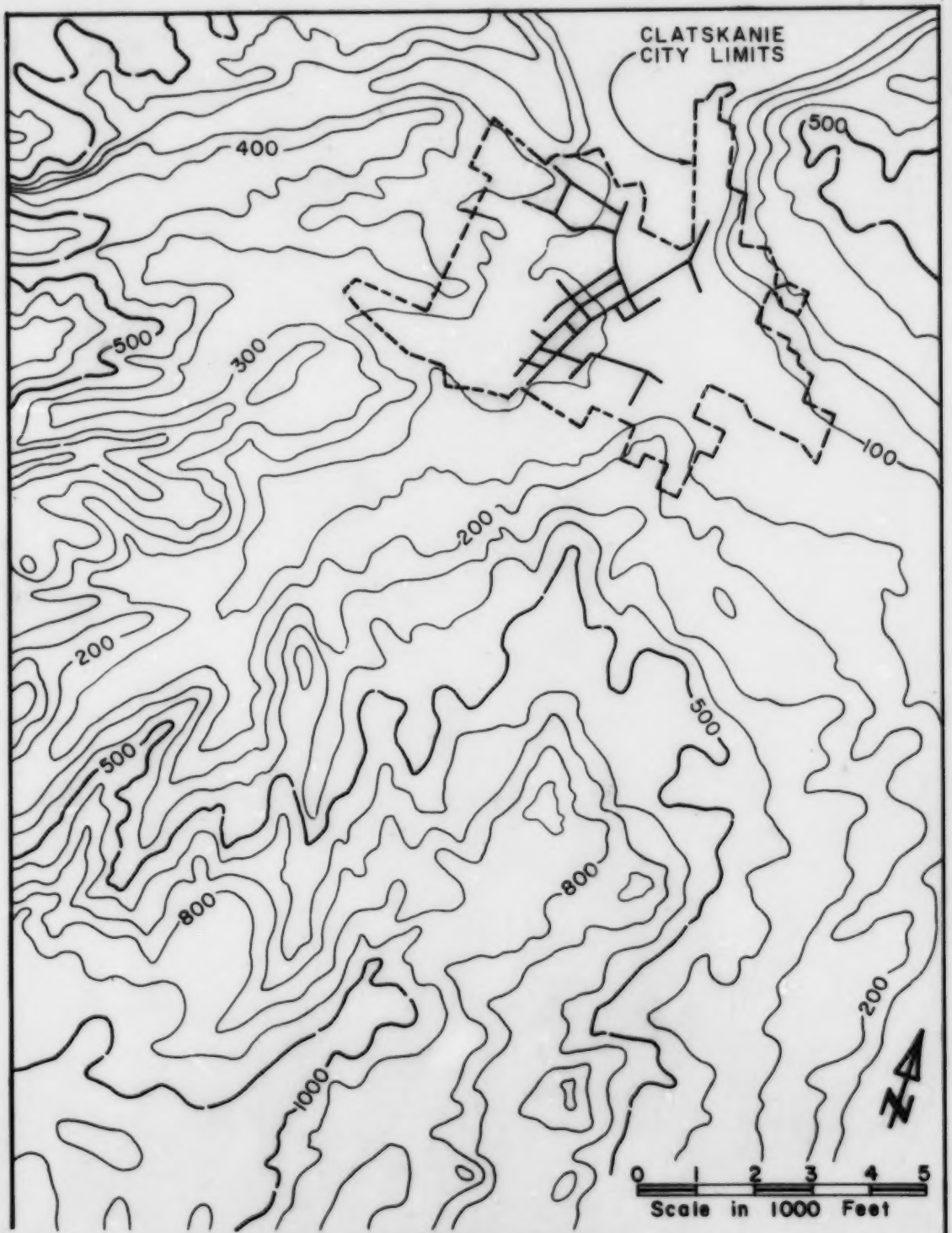


Figure 1: Clatskanie Topographic Features

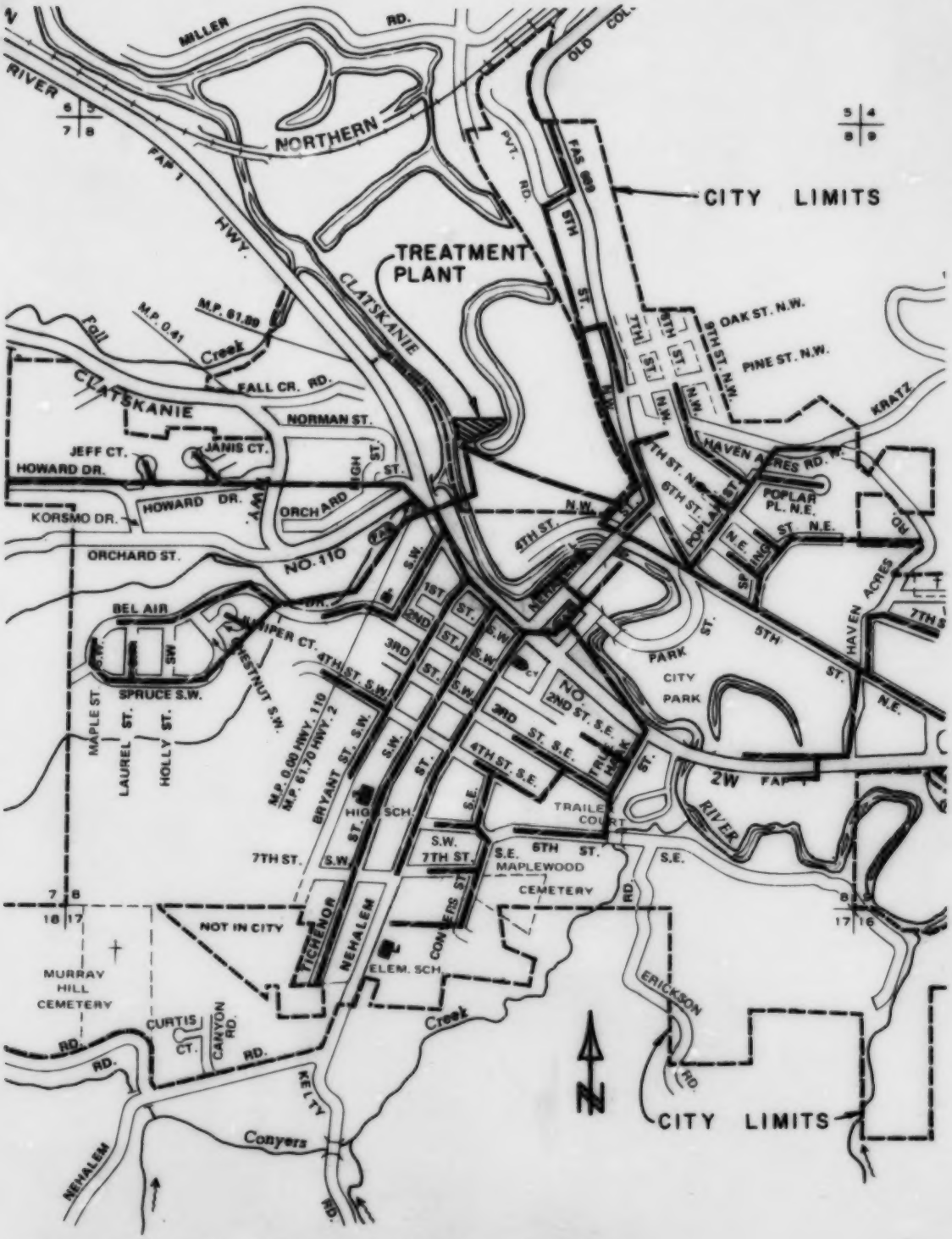


Figure 2: Clatskanie Sewer System

The areas south of the river are transmitted to the main pump station, where it is pumped across the river to the treatment plant.

In 1974 an I/I study was conducted on the Clatskanie collection facilities. This study showed that the system was subject to heavy I/I, resulting largely from cracked and settled pipes, defective joints, leaky manhole covers and similar causes. It was also found that during major storm events existing lift stations were overloaded and that untreated wastewaters were bypassed to the Clatskanie River at five locations.

Field measurements of six storm events in early 1974, coupled with a comparison of historical records, indicated that major storm events could be expected to yield total sewer flows exceeding one mgd (3785 m3/day). Several alternatives were studied for controlling discharges contributed during wet weather. One procedure involved reduction of I/I to a level where the existing treatment facility could handle the entire wastewater flow anticipated. Other techniques investigated involved the application of various treatment alternatives having capacity large enough to process full wet weather flow, as well as dry weather flow. Cost effectiveness analysis led to the selection of a joint dry/wet weather flow treatment facility employing DAF-Contact Stabilization activated sludge, as described in the following sub-section.

During the course of the demonstration project, it was found that wet weather flows often approached 1.5 mgd and, over short periods of time, significantly higher flows occurred. On January 12, 1980, it is estimated that an instantaneous peak flow of 3.1 mgd was generated by the system. At this time 2.3 mgd was flowing through the treatment plant and the remaining 0.8 mgd was bypassed. This, of course, was an instantaneous peak and it is estimated that in the 5.25 hour period during which bypassing occurred, the average bypass flow was 0.4 mgd (1514 m3/day). The total volume bypassed on this date was estimated to be 87,500 gallons (331 m3). At the time it was impossible to actually observe the bypasses since the river level was high and the bypass outlets were submerged. Bypass outlets are fitted with flap gates. The bypass flow estimates were developed by analyzing pump station flow records and they are very rough. This apparently was the only occurrence of bypassing during the two year demonstration project.

No direct inflow sources are known to exist in the Clatskanie system. River levels were monitored during two storm events, and no correlation between river level and plant flow rate could be determined.

Treatment Plant

The joint dry/wet weather flow treatment facility at Clatskanie is constructed on a 1 acre (.405 hectare) site located near the City's central business district. The site is adjacent to the Clatskanie River and is protected from flooding by earthen dikes approximately 15 ft (4.6m)

high. Figure 3 shows a simplified layout of the facility, and photographs are shown in Figure 4. Figure 5 gives a simplified process schematic.

Pretreatment: Raw sewage from lift stations 1 and 2 (See Figure 2) enters the headworks of the plant and is pretreated in a 9400 gal (36 m3) aerated grit chamber followed by coarse screening and comminution in a 12 ft x 2 ft x 3.5 ft (3.7m x 0.6m x 1.1m) channel. Collected grit is washed and sent to land disposal. A parallel channel with a second bar screen acts as a bypass in the event of mechanical failure of the comminuting device. See Figure 4a.

Primary Clarification: Following pretreatment, wastewater flows by gravity to a circular primary clarifier having nominal dimensions of 6.5 ft x 20 ft diam (2m x 6.1m diam). This treatment component is shown in Figure 4b.

The primary clarifier is designed to operate in a conventional sedimentation mode during dry weather and other low flow conditions. With high wet weather flows, when traditional surface loading rates are exceeded, the clarifier is converted to a dissolved air flotation (DAF) mode. As shown in Figure 5, a primary overflow bypass line is available for use with extreme flows, when the capacity of the DAF unit is exceeded. Flows in excess of 1.25 mgd (4731 m3/day) were generally bypassed to the secondary.

In the conventional sedimentation mode pretreated wastewater is introduced into a center feedwell. The top of the feedwell is located approximately 1.5 ft (.46 m) below the surface of the water and is sealed at the bottom as a design requirement for DAF operation. Influent flow is distributed evenly to the clarifier settling zone by discharge over the top of the center well. Turbulence within the centerwell prevents accumulation of solids during operation.

Primary settled liquor discharges over a peripheral v-notched weir, where it is collected in an effluent launder and then sent by gravity to secondary treatment. Settled primary sludge is raked to a bottom hopper and then drained to a sludge sump, where it is temporarily stored prior to further processing. During conventional settling, operation of the bottom rake and an integral surface scum skimmer is normally intermittent, as dictated by daily requirements at the plant. The system is driven by a variable speed drive permitting rates between 0.27 and 1.10 rpm.

When high wet weather flows are imminent, the primary clarifier is switched to the DAF mode. First, accumulated primary settled sludge is withdrawn from the clarifier bottom zone and discharged to the sludge sump. The air pressurization system is then turned on to initiate float formation. During start-up of the flotation system the surface skimming mechanism and sludge rakes are not operated.

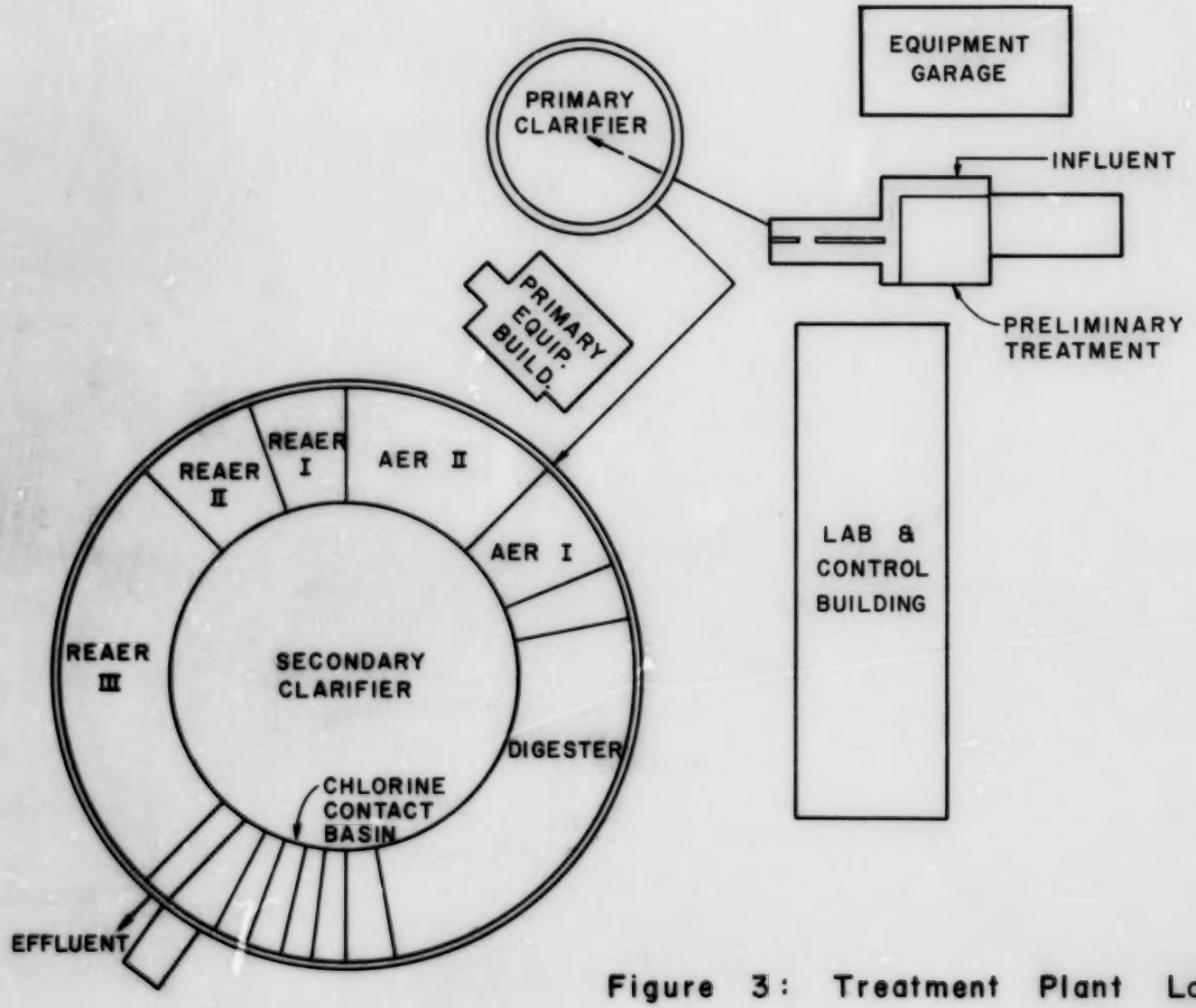
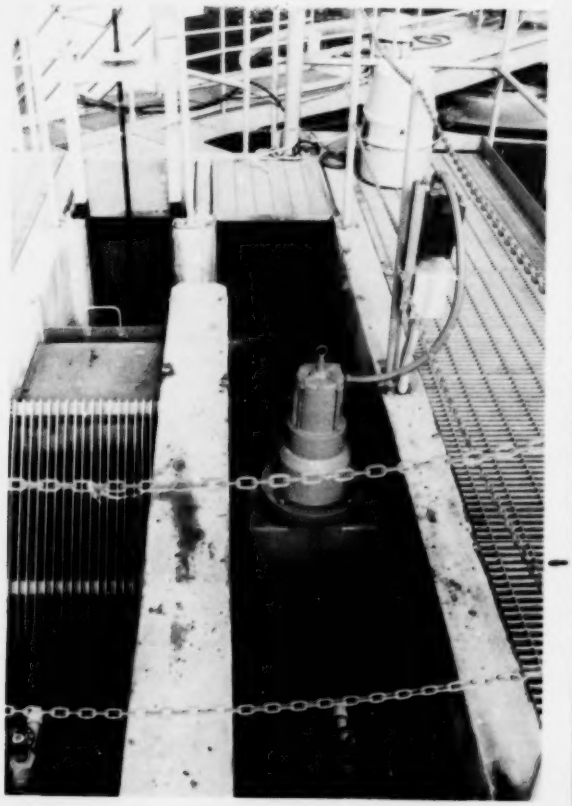


Figure 3: Treatment Plant Layout



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4b.

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Figure 4: Treatment Plant Photographs

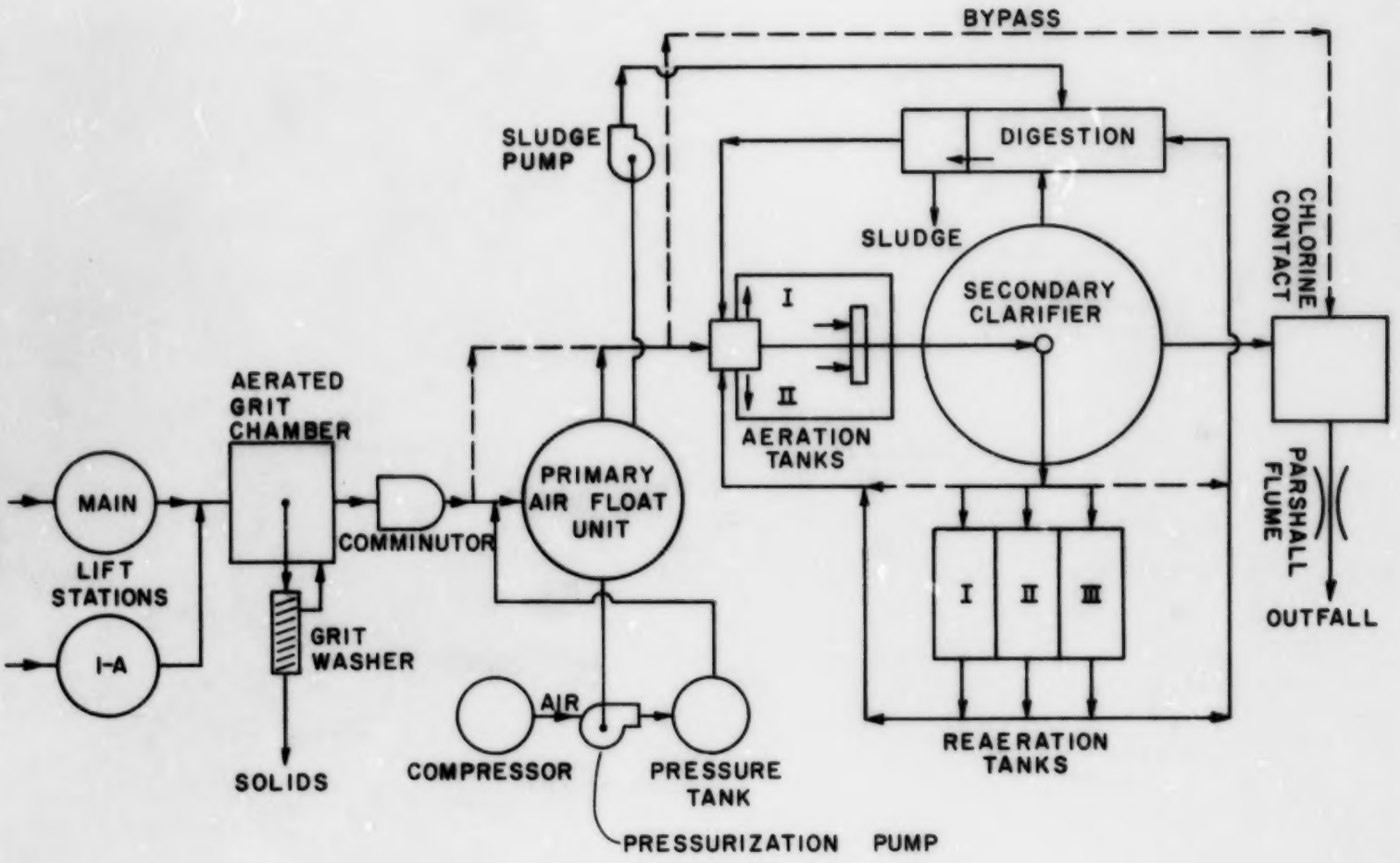

Figure 5: Basic Flow Diagram

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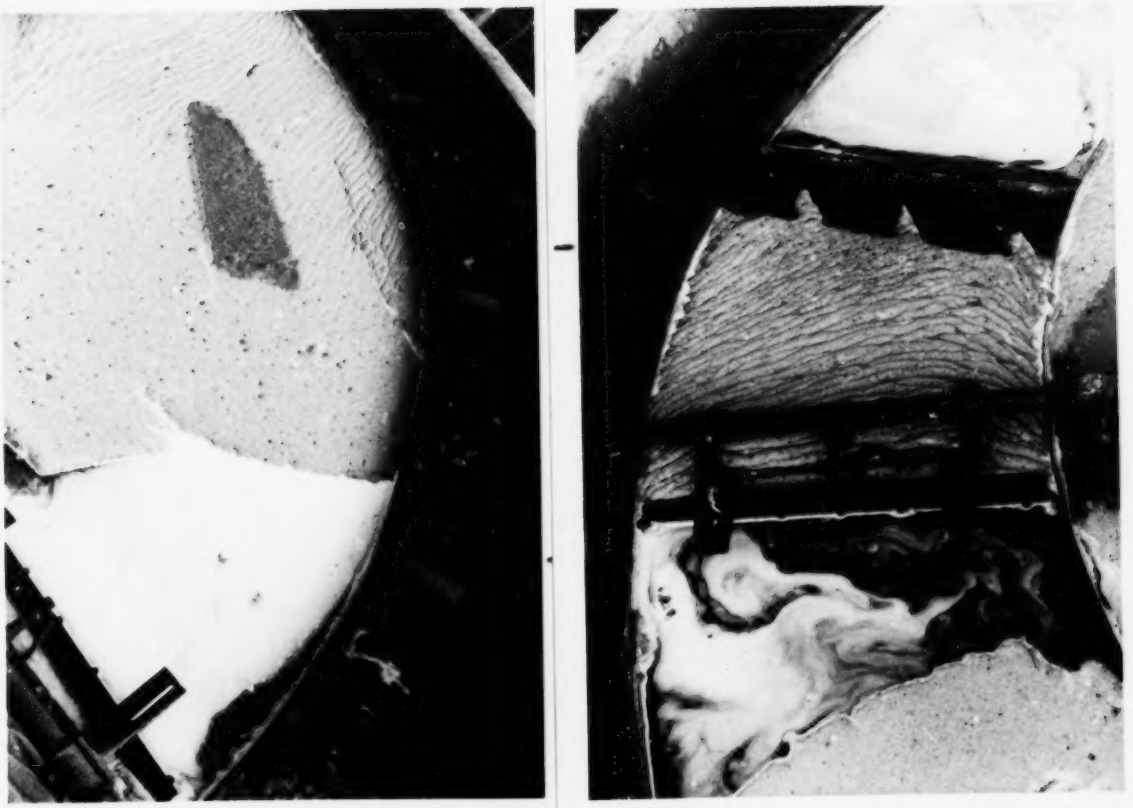

6a. Float formation

6b. Float discharge

Figure 6: Photographs of DAF Operation.

Once a float has formed the integrated sludge rake/surface skimmer mechanism is turned on. As with the conventional settling mode this operation is intermittent, but cycling time and rate are tailored to optimze quality of the float. Figure 6 shows photographs of DAF operation indicating float quality during the skimming cycle (Figure 6a) and as floated solids are discharged to a float sump (Figure 6b). Floatable solids collected in the float sump and any non-floatables which settle to the bottom of the clarifier are ultimately discharged to the primary sludge sump where they are temporally stored for further processing.

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Air injection rates for the DAF system are variable, with most operation being between 30 and 90 scfh (0.85 and 2.6 m3/hr). Air injection pressures and flow rates are also variable but are interdependent. Because of the head characteristics of the two stage pressurization pumps, pump pressure will rise as recirculating flow is reduced. To compensate for this and maintain available flotation air, air pressure must be increased. Typically, during DAF operation recirculating flows are maintained between 100 and 150 gpm (9.1 and 13.7 m3/day), with air pressure settings between 70 and 60 psig, (482 and 414 kN/m2), respectively.

The air pressurization system used in the DAF mode of operation (See Figure 5) employs two pumps in series, an air compressor, a saturzation tank and associated controls. In operation, a portion of primary clarifier liquor is withdrawn from the effluent launder and sent through the two stage pressurizing pumps where compressed air is injected into the inter-stage piping. Air-liquor mixture then passes to a pressurizing tank for air dissolution and is finially combined with incoming pretreated wastewater in the center feedwell to the clarifier. Release of air pressure in the clarifier generates fine bubbles which attach to solid particles floating them to the surface. The bottom-sealed center feedwell acts as a mixing zone to promote air/solids contact and to also minimize direct settling before flotation can be developed.

Secondary Process: The third component of the Clatskanie treatment plant (see Figure 3) consists of the activated sludge system, secondary clarifier, aerobic digestion facilities and chlorine contact chamber, all constructed within a circular configuration. This component has overall nominal dimensions 12 ft x 75 ft diam (3.7m x 23 diam) with a 45 foot diameter (13.7m) central secondary clarifier. The annular volume surrounding this central clarifier is divided into multiple compartments for mixed liquor contact and/or aeration, for settled solids reaeration, for waste primary and secondary sludge digestion (aerobic), and for chlorine contact.

The contact/aeration and sludge reaeration compartments are further subdivided into operating zones to provide flexibility for the wide range of present-day and future design flows anticipated. Two mixed liquor contact/aeration zones are provided having volumes of 14300 gal (54 m3) and 29300 gal (111 m3), or a combined mixed liquor volume of 43600 gal (165 m3).

For settled solids reaeration (stabilization) three operating zones are provided. Two reaeration zones are of similar size, having volumes of 12700 gal (48 m3) and 16000 gal (61 m3), while the third is larger and has a capacity of 59300 gal (224 m3). Piping arrangements for the secondary process are flexible to allow for the use of any combination of contact/aeration and reaeration volumes needed to maintain contact stabilization process performance. In addition, these piping arrangements will allow for operation as a complete-mix activated sludge facility under

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low hydraulic and process loadings which occur during present-day DWF conditions.

Aerobic sludge digestion facilities consist of a 60100 gal (227 m3) aeration/digestion zone and a 6900 gal (26 m3) stilling well where digested sludge is decanted prior to land disposal. The chlorine contact zone is fitted with five longitudinal baffles providing an effective length-to-width ratio of approximately 4:1. Chlorine gas is injected at the up-stream end of the contact chamber on a flow proportional basis, with dosage rates defined by monitoring chlorine residuals.

Low pressure air for the biological processes is provided via three centrifugal blowers with minimum ratings of 650 scfm (18 m3/hr) at seven psi (48 kN/m2). Air lift pumps are provided for settled secondary solids return, for reaeration solids return, for digested sludge decant liquor and for secondary clarifier scum return. Diffusers provide coarse bubble aeration for mixing and oxygen transfer.

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SECTION 5

DESIGN FEATURES AT CLATSKANIE

This section reviews the conceptual basis for treatment design at Clatskanie and describes features pertinent to development of the research and demonstration study. Literature applicable to system design is also summarized and discussed.

General

The joint dry/wet weather treatment facility at Clatskanie has been designed to process a range of flows between 0.13 and 1.25 mgd (492 and 4731 m3/day). This range represents a flow variation of 10:1 based on the ratio of design wet weather storm flow to a current minimum daily flow of 0.13 mgd (492 m3/day), or approximately 6:1 based on the ratio of design stormflow to a current average daily flow of 0.2 mgd (757 m3/day). This wide variation in anticipated wastewater flows, coupled with concomitant changes in raw waste BOD (5-day, Biochemical Oxygen Demand) and TSS (total suspended solids) loads during WWF, led to the conceptual development, design, and ultimate construction of the facility demonstrated in this study. Flows exceeded 1.25 mgd (4731 m3/day) once each year during the two year demonstration study.

As noted previously in Section 2, the design concept at Clatskanie employs a dual-use primary clarifier capable of DAF operation with high flows in conjunction with a contact stabilization activated sludge system having flexible volumetric capacity to allow processing of highly variable in-coming loads. This concept was selected after considering effluent requirements at Clatskanie and individual process capabilities needed to meet these requirements. Subsequent parts of this section review the use of flotation and contact stabilization systems in wastewater treatment and discuss the application of these processes at Clatskanie.

Air Flotation Treatment

Flotation systems have been utilized for many years in industrial process applications (2,3). However, it has only been in the last two decades that this unit process has been employed specifically in pollution control situations for clarifying wastewaters (4-11), for sludge thickening (12-18), and for algal harvesting (19, 20). Application to the control of combined sewer overflows and other wet weather discharges is even more recent (21-26).

Three methods have been developed to achieve flotation of suspended material -- gravity flotation, vacuum dissolved air flotation and pressure dissolved air flotation (DAF). Of these, (pressure) DAF is the most widely employed in waste treatment applications, due in part to more effective solids removal and to low operating sensitivity (27). In many applications chemical additives have been used to induce and/or improve flotation (21, 22, 24, 26, 28). Extensive discussions of DAF process theory, design requirements and operating modes can be found in references 29 to 35.

The first study of DAF treatment of combined sewer overflow (CSO) was completed in 1970 (21). This study investigated performance characteristics of a treatment train consisting of screening, DAF and hydrocyclones. With all separatory equipment utilized in the train, TSS and BOD removals of 69 and 40 percent were achieved. However, significantly lower TSS removals were found with sequential screening/DAF, alone. Chemical additives improved treatment efficiencies in most cases reported.

In another study (22), a 5 mgd (18925 m3/day) combination screening/DAF demonstration system was designed, constructed and evaluated. Treatment of 55 combined sewer overflows in conjunction with chemical addition yielded TSS removals in the range of 65-80 percent. BOD and COD removals of 55-65 percent were obtained. Without the use of chemical flocculants, removals of these constituents were in the range of 40-50 percent.

A third evaluation studied screening and screening/DAF to treat a total of 58.5 mgd (222,000 m3/day) of CSO in Racine, Wisconsin under full-scale conditions. Based on operating results from two screening/DAF systems, it was concluded that removal efficiencies of 60 to 75 percent can be expected for suspended solids and 50 to 65 percent for BOD. Screening alone can be expected to achieve 50 percent suspended solids removal and 20 percent BOD removal.

A fourth study (24) used laboratory and pilot-scale systems to develop DAF process data for the design of a full-scale, 24 mgd (90840 m3/day) prototype facility treating combined sewer overflows. After construction the prototype facility was tested and evaluated under WWF conditions over a period of 10 months. Surface loading rates varied from 2500 to 5700 gpd/sq ft (102 to 232 m3/m2/day), with alum dosages from 0-300 mg/l. The most desirable performance occurred at a loading rate of 3600 gpd/sq ft (147 m3/m2/day) and an alum dosage rate of 75 mg/l. Under this condition, TSS and BOD removals equalled 51 and 82 percent, respectively. Other aspects of the design, construction and evaluation are described in the referenced report.

Contact Stabilization Treatment

Conceptually, the contact stabilization mode of activated sludge treatment relies on the sorptive ability of biological solids to remove biologically oxidizable organic matter with short retention periods (27, 29). When properly designed to take advantage of this short-retention, bio-contact without deterioration of effluent quality significant reductions in overall process volumes can be achieved relative to other modes of activated sludge treatment. However, as retention periods needed for treatment are increased, volumetric capacity required also increases, and thus the process loses its economic advantage (27). Contact stabilization has been applied to treatment of industrial wastes as well as to treatment of domestic wastewaters (29).

The exact mechanism of organic removal by activated sludge in a short-contact mode was the subject of controversy for many years. Although the effectiveness of the process for removing particulate organics was generally agreed upon (36-40), its response for soluble organics removal was not. Studies eventually confirmed that bio-enzymatic activity is a prerequisite for effective uptake of soluble organics in the process (40-45).

The conceptual basis for contact stabilization treatment, namely, removal of oxidizable organics in a short detention time configuration, coupled with the ability of the process to adjust to variations in loadings and flow rates, makes it an attractive alternative for treatment of wet weather flow discharges. Recognizing this potential advantage the U.S. Environmental Protection Agency and the City of Kenosha, Wisconsin co-sponsored a study to demonstrate and evaluate the performance of the contact stabilization process for the treatment of combined sewer overflows (46). The study represents the principal work to date in the application of contact stabilization activated sludge for CSO control.

The demonstration work at Kenosha involved the design, construction and evaluation of a 20 mgd (75700 m3/day) pilot contact stabilization system which was installed at the site of an existing DWF facility employing the conventional activated sludge mode of operation. The pilot contact stabilization system was put into operation only during WWF periods.

Results from the demonstration project showed that contact stabilization treatment of CSO's is a feasible concept. For the configuration evaluated optimum ranges of the principal process variables were as follows:

- MLSS Concentration > 2100 mg/l
- Reaeration Time 1-4 hours

- o Stabilization Time (Static) < 5 days
- o Contact Time (with recyle flow) > 10 minutes

In this project reaeration time refers to the retention time in the stabilization tank during operation of the system for treatment of overflows from a storm event; stabilization time refers to the length of the waiting period between overflow events, during which time biosolids in the stabilization tank are held under aeration on a standby basis.

For thirty runs, treating a total of 106.5 million gallons (403,103 m3) of wastewater, arithmetic mean percentage removals of TSS, BOD and total organic carbon (TOC) were 90.4, 84.8 and 76.5, respectively. Arithmetic mean effluent concentrations were 23 mg/l, 16 mg/l and 23 mg/l for TSS, BOD and TOC, respectively.

Extensive studies conducted on the sludge in the stabilization tank revealed that 5 to 7 days (beyond the sludge age of the DWF plant) appeared to be the maximum duration that the sludge could be held in the tank in a static condition and still be useful in the demonstration system. However, if waste activated sludge were continually fed to and withdrawn from the stabilization tank, the sludge could remain in a viable condition indefinitely. Additional data on costs of WWF treatment and on interactions between the WWF and DWF treatment plants are provided in the referenced report.

Application at Clatskanie

The DAF/Contact Stabilization treatment system at Clatskanie was developed to meet four principal design constraints for DWF and WWF conditions:

- o Process capacity to handle raw waste flows between 0.13 and 1.25 mgd (492 and 4731 m3/day) in a single facility;
- o Treated effluent quality not to exceed 20 mg/l of BOD and TSS, based on arithmetic mean monthly values;
- o Wide variations in raw waste BOD and TSS concentrations during wet weather flows; and
- o Sufficient process flexibility to meet flow and load variability intermediate to the extremes anticipated.

Table 1 summarizes the design criteria used in the development of the Clatskanie facility. To meet a design average daily DWF range of 0.2 to 0.5 mgd (757 to 1892 m3/day) the primary clarifier at Clatskanie was sized to operate at nominal sedimentation overflow rates of 640 to 1590 gpd/sq ft (26 to 65 m3/m2/day). With an effective liquid depth of 6.5 ft (2m), this allows for nominal design hydraulic retention times between 0.7 and 1.8 hours. In this study, nominal overflow rates and retention times refer to calculated values relating flow rate and area or volume.

Table 1 : Summary of Design Criteria Used at Clatskanie

Criteria		Present-day DWF	Present-day WWF	Future DWF	Future WWF
Population		1540	1540	3900	3900
Design Flow, mgd		0.2	1.0	0.5	1.25
Raw Waste BOD,	mg/l	186	-	186	-
	lb/day	308		780	
Raw Waste TSS,	mg/l	232	-	232	-
	lb/day	385		975	
Primary Clarifier					
Design Mode		Sed.	DAF	Sed.	DAF
Overflow Rate, gpd/sq ft		640	3185	1590	3980
Nominal Depth, ft		6.5	6.5	6.5	6.5
Activated Sludge					
Design Mode*		CM	CS	CS	CS
Aeration Solids, mg/l		2500	2500	2500	2500
Reaeration Solids, mg/l		-	12000	12000	12000
Min. Aeration Time, hr		3.5	-	-	-
Max. Aeration Time, hr		5.2	-	-	-
Min. Contact Time, min.		-	22	41	16
Max. Contact Time, min.		-	63	84	50
Reaeration Time, hr		-	1-4	3-6	1-4
Clarifier Ov. Rate, gpd/sq ft		126	630	315	790
Nominal Depth, ft		12	12	12	12
Aerobic Digester					
Available Volume, gal		60100	60100	60100	60100
Digestion Time		Variable	Variable	Variable	Variable
Chlorine Contact Tank					
Volume, gal		26400	26400	26400	26400
Contact Time, hr		3.2	0.63	1.27	0.51

Note: mgd x 3785 = m3/day; lb x 0.4536 = kg; gpd/sq ft x 0.041 = m3/m2/day; ft x 0.3048 = m; gal x 0.003785 = m3

*CM = Complete-mix; CS = Contact Stabilization

Above 0.5 mgd (1982 m3/day) the primary clarifier is designed to operate in the DAF mode. Between this lower flow limit and the upper design limit of 1.25 mgd (4731 m3/day) nominal DAF overflow rates (excluding recycle flow) vary between 1590 and 3980 gpd/sq ft (65 and 162 m3/m2/day). Design nominal hydraulic retention times for this flow regime vary between 18 and 44 minutes, excluding recycle flow. Design flow and overflow rate relationships for primary clarification are summarized in Figure 7.

The secondary clarifier at Clatskanie was designed to provide a nominal hydraulic retention time and a nominal overflow rate (excluding recycle flow) of 2.9 hours and 790 gpd/sq ft (32 m3/m2/day), respectively at the maximum design raw waste flow of 1.25 mgd (4731 m3/day). The maximum design overflow rate of 790 gpd/sq ft (32 m3/m2/day) was selected in order to provide secondary clarification capability for suspended solids removal during sustained periods of wet weather flow. The relationship between secondary clarifier nominal overflow rate and raw waste flow is also shown in Figure 7.

The contact stabilization system at Clatskanie was designed for operating flexibility to meet the wide range of dry and wet weather flows and loads anticipated under both present day and future conditions. Traditional DWF criteria for contact stabilization design require contact/aeration times between 0.5 and 3 hours, with reaeration times of 3-6 hours (27, 29). However, in the demonstration study at Kenosha, Wisconsin (46) contact and reaeration times as low as 0.17 hours (10 minutes) and one hour, respectively, were found to provide acceptable treatment of combined sewer overflows. Both DWF and WWF criteria noted above were used as guidelines for the design at Clatskanie, since both flow regimes must be processed in the same facility.

As noted previously in this report (Section 2), the mixed liquor contact/aeration and secondary solids reaeration tankage are subdivided into operating compartments or zones to allow flexibility for meeting the wide range of treatment conditions expected. Figure 8 summarizes nominal retention time as a function of flow rate for contact/aeration using operating zones 1 and 2, individually, as well as in combination. Since actual nominal retention time is dependent on recycle flow rate, two sliding scales are shown illustrating the actual raw waste flows which can be processed at recycle rates of 0.1 and 0.5 mgd (378 and 1892 m3/day).

Reaeration time is also dependent on recycle flow rate. Reaeration capacity is subdivided into three operating zones which can be used independently or in combination, thus providing a total of seven distinct reaeration volumes and associated ranges of reaeration times.

Table 2 summarizes reaeration zones and volumes which are useable for recycle flows between 0.1 and 0.5 mgd (378 and 1892 m3/day) based on both traditional and wet weather flow criteria for reaeration time. Obviously, operational choice of reaeration volume and recycle flow must be balanced to maintain a desired reaeration time. This selection must also be

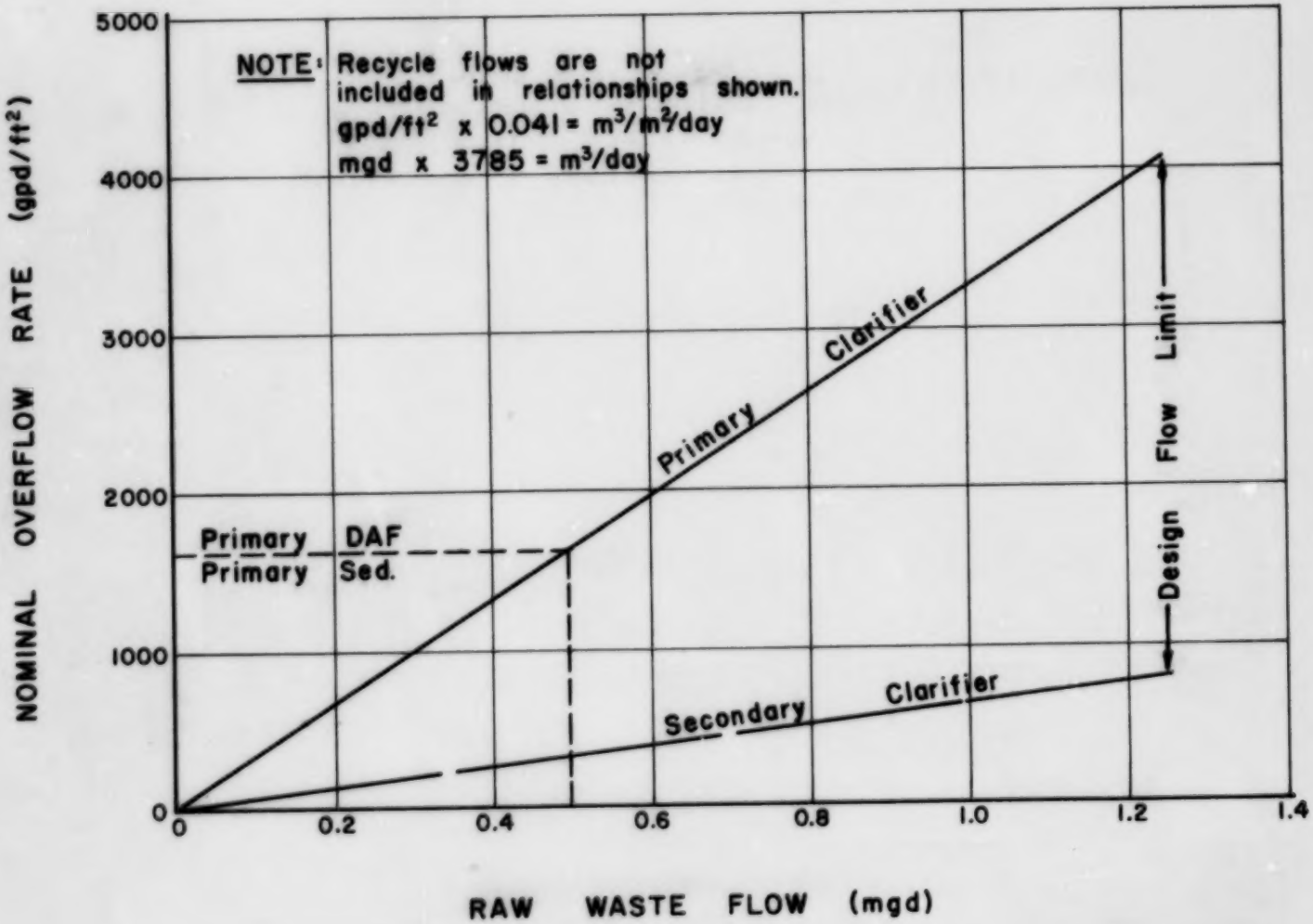

Figure 7: Overflow Rate and Raw Waste Flow Relationships

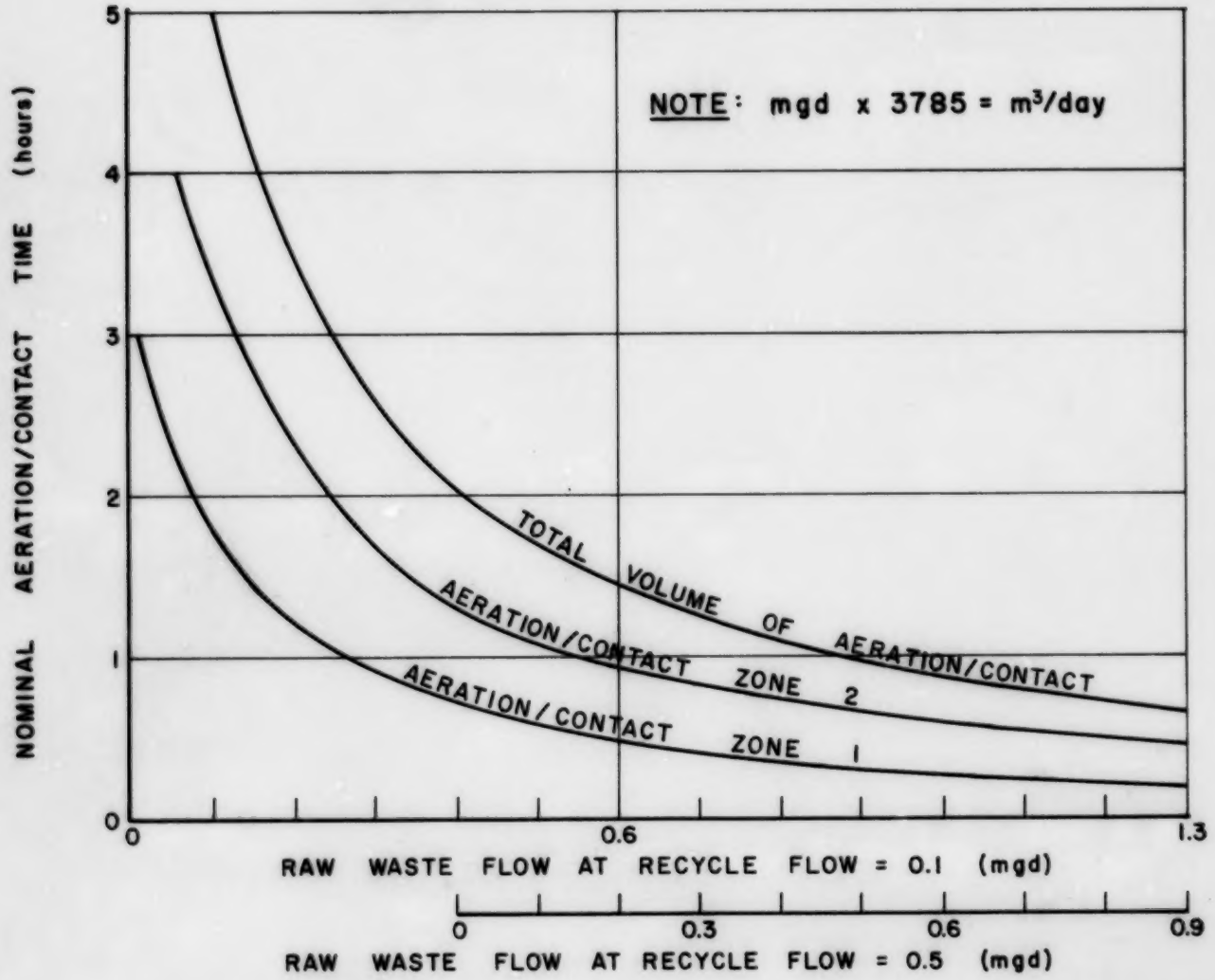

Figure 8: Aeration/Contact Time & Raw Waste Flow Relationships

Table 2: Reaeration Operating Volumes at Clatskanie*

Recycle Flow (mgd)	Basis	Reaeration Volume (gal) Max	Min.	Reaeration Zone(s) Useable 1	2	1+2	3	1+3	2+3	1+2+3
0.10	T	25000	12500	X	X					
	W	16667	4167	X	X					
0.15	T	37500	18750			X				
	W	25000	6250	X	X					
0.20	T	50000	25000			X				
	W	33333	8333	X	X	X				
0.30	T	75000	37500				X	X		
	W	50000	12500	X	X	X				
0.50	T	12500	62500					X	X	X
	W	83333	20833			X	X	X	X	
Actual Reaeration Volume (1000's gal)				12.7	16.0	28.7	59.3	72.0	75.3	88.0

*Note: T indicates traditional design criteria; reaeration time = 3-6 hours
W indicates WWF design criteria; reaeration time = 1-4 hours
gal x 0.003785 = m3 mgd x 3785 = m3/day

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integrated with contact zone criteria and with other requirements in the biological process (e.g. biosolids settleability and return capacity) to ensure acceptable treatment.

Design mixed liquor suspended solids (MLSS) and reaeration solids levels of 2500 mg/l and 12000 mg/l, respectively, were selected at Clatskanie (see Table 1). For these levels contact/aeration solids capacity will vary between 298 and 909 pounds (135 and 412 kg), while reaeration solids capacity will vary between 1271 and 8807 pounds (576 and 3995 kg).

Design contact, reaeration and total process biosolids capacities for several operating configurations are summarized in Table 3. This tabulation also shows the range of design input BOD loads which can be treated in each operating configuration for process loadings between 0.15 and 0.3 kg-BOD/day/kg-biosolids. Design loading capacity is provided for expected WWF contributions as well as for DWF contributions.

Chlorination facilities at Clatskanie have been designed to provide 30 minutes contact at the maximum WWF of 1.25 mgd (4731 m3/day). Chlorination contact times at other flow rates are summarized in Table 1.

Table 3: Design Process Conditions for Several Operating Configurations*

Contact/Aeration Basin			Reaeration Basin			Process	Input BOD, in lb/day, for F:M Ratios of 0.15 to 0.3 kg BOD/day/kg-Biosolids
Zone	Vol. (gal)	Biosolids (lb)	Zone	Vol. (gal)	Biosolids (lb)	Biosolids (lb)	
1	14300	298	1	12700	1271	1569	235-471
1	14300	298	2	16000	1601	1899	285-570
1	14300	298	1+2	28700	2872	3170	476-951
2	29300	611	1+2	28700	2872	3483	522-1045
2	29300	611	3	59300	5935	6546	982-1964
1+2	43600	909	3	59300	5935	6844	1027-2053
1+2	43600	909	1+3	72000	7206	8115	1217-2434

*Note: gal x 0.003785 = m3; lb x 0.4536 = kg

SECTION 6

STUDY DESIGN

Objectives

The research and demonstration study at Clatskanie was designed with five principal objectives in mind:

- To demonstrate effluent control capabilities of the joint dry/wet weather flow system;
- To define and evaluate performance characteristics of individual process components;
- To identify parameters and conditions affecting process performance;
- To present cost data for joint dry/wet weather treatment; and
- To develop comparisons for nation-wide application of the treatment concept demonstrated.

To meet these objectives a field survey and data collection program was developed to document plant performance, to assess operational procedures used at the plant, to evaluate biotreatment responses, and to provide cost information for the treatment works. The scope of the study program is described in this section along with specific techniques and procedures used for storm reconnaissance, for sample collection, for laboratory analysis and for data interpretation.

Scope and General Approach

The scope of the study program was developed to obtain several kinds of information related to the Clatskanie treatment facility. The kinds of information obtained include the following:

- A definition and analysis of rainfall and plant flow patterns observed during storm events sampled in the investigation;
- A definition of the magnitude and variability of secondary effluent BOD and TSS values discharged from the plant;

- A definition of overall TSS and BOD removals achievable by the plant;
- A definition of performance removals achievable by primary sedimentation and primary DAF during both DWF and WWF periods;
- An identification of contact stabilization treatment capabilities under WWF conditions;
- An identification of operational procedures and design parameters which affect DAF primary clarification at the plant;
- An identification of operational procedures and design parameters which affect contact stabilization treatment at the plant;
- A definition and evaluation of biosolids viability effects for contact stabilization treatment of WWF;
- A collation of supplementary information on nutrients, heavy metals, BOD/COD ratios and similar analyses applicable to plant performance and/or effluent discharge;
- A summary of capital and operating costs for the treatment facility with a comparison to conventional modes of treatment; and
- An evaluation and comparison of results obtained in the investigation relative to potential nation-wide application of the treatment concept demonstrated.

Within this general study scope, several approaches for developing a field sampling program at the treatment facility were explored at the outset of the investigation. One potential approach involved establishing a rigid factorial experimentation. A second approach involved identifying key treatment plant variables likely to affect process performance, allowing these to vary randomly during the field sampling program, and then applying multiple regression techniques to separate the influence of each interacting variable (47, 48). Both of these statistically-based approaches to study design were rejected because of uncertainty in storm characteristics anticipated during progress of the study, because of practical considerations related to implementing these programs within budgetary constraints, and, in the case of the second approach (47), because of potential limitations in extrapolating the results obtained to nation-wide applications.

The general approach ultimately selected for the study is based on a functional understanding of individual treatment process components, along with statistical analysis of data collected in the field sampling surveys. These field surveys were designed to be responsive to a wide range of anticipated rainfall and plant flow conditions and were also

developed to document plant performance characteristics under varying modes of operation and for various ranges of design and operational variables. An integral part of this work involved establishing baseline conditions for DWF operation, from which WWF comparisons could be generated. Criteria used in planning for and performing these surveys are discussed below:

1. Plant Flow Regimes: From a review of historical rainfall and plant flow patterns for the treatment facility, two storm flow regimes were identified for use in program planning. Although it was recognized from the outset that other patterns would most likely be experienced, these regimes served as a convenient starting point for designing the field surveys.

The first storm flow regime is representative of flow conditions which result from the occurrence of storms of relatively short duration (0 to 2 days). These storms are often intense and produce a plant flow pattern which increases to a peak (or multiple peaks) well within 24-hours of rainfall initiation, with a return to the DWF regime shortly thereafter.

The second storm flow regime is representative of a northwest Oregon weather pattern which yields generally low intensity rainfall for periods of days to weeks, interspersed with periods of heavier precipitation from coastal storms. This type of weather pattern can produce sustained high WWF conditions at the treatment plant with multiple flow peaks caused by the periodic high intensity storms. For this type of plant flow regime, process operation must allow for sustained treatment at flows which can be as much as 6-10 times those experienced under DWF conditions. Dry weather flow, itself, is generally within the range of 0.13 to 0.3 mgd (492 to 1136 m3/day).

2. Primary Clarification: As noted in Section 3 (Table 1), the primary clarifier has been designed to operate in the plain sedimentation mode up to 0.5 mgd (1982 m3/day), based on average daily flow. This flow rate was also selected as the principal study criterion for converting from primary sedimentation to DAF operation in response to storm events. Exceptions to this occurred only for those surveys which were designed to document baseline performance for DAF operation at low overflow rates (DWF conditions) and for plain sedimentation operation at high overflow rates with flows in excess of 0.5 mgd (1982 m3/day).

During plain sedimentation, properties of incoming suspended solids, as well as such design-related variables as overflow rate, flow variability, weir loading and maintenance, and inlet conditions may affect actual performance as measured in the field. During DAF operation these factors plus air injection rate, air pressure, recycle flow rate, and skimmer speed can affect measured removals of suspended solids.

For this study, field surveys were designed principally to document process TSS and BOD removals relative to overflow rates observed as a result of variable incoming waste flows. Surveys were also designed, however, to define the influence of several DAF operating variables on overall treatment characteristics.

3. Complete-mix Activated Sludge: The secondary process has been designed with sufficient flexibility to permit operation in the complete-mix activated sludge mode under present-day DWF conditions. Although not a major component of the study scope, some performance data were obtained with this mode of operation.

4. Contact Stabilization-Hydraulic Criteria: The study scope for field surveys designed to document contact stabilization performance was selected principally on the basis of WWF criteria obtained in the study at Kenosha, Wisconsin (46). As noted previously in Section 3, this study found that acceptable contact stabilization treatment of combined sewer overflows could be achieved with mixed liquor suspended solids concentrations greater than 2100 mg/l, nominal contact times (including recycle flow) greater than 10 minutes, and nominal reaeration times in the range of 1-4 hours. Viability of the biosolids held in the process was also found to be a major factor in obtaining effective treatment. In this part of Section 4 hydraulic criteria applicable to developing study scope are discussed. Biosolids criteria for developing study scope are presented subsequently.

At the outset of the investigation several important differences between the Kenosha study and that at Clatskanie were recognized. First, at Kenosha the storm events studied were relatively short-term, lasting approximately 2 to 12 hours, while in the Clatskanie study these events were expected along with long-term events lasting days or weeks. Study scope was designed to respond to both types of events.

A second difference between the two investigations involves system deployment. At Kenosha, two separate treatment systems were used for processing DWF and WWF events, while at Clatskanie, both DWF and WWF are treated in a single facility. Another difference relates to system operation. At Kenosha the system studied was operated intermittently to treat only combined sewer overflows from storm events. The Clatskanie system, on the other hand, operates continuously in response to all flow situations. Finally, the Kenosha contact stabilization system did not employ primary clarification during treatment of storm overflows. At Clatskanie all flow is treated by either DAF or plain sedimentation prior to transport to the integrated secondary process.

The implications of using the Kenosha WWF hydraulic criteria as guidelines for developing the scope of contact stabilization field surveys at Clatskanie, are summarized in Figures 9 and 10. In Figure 9 the plant

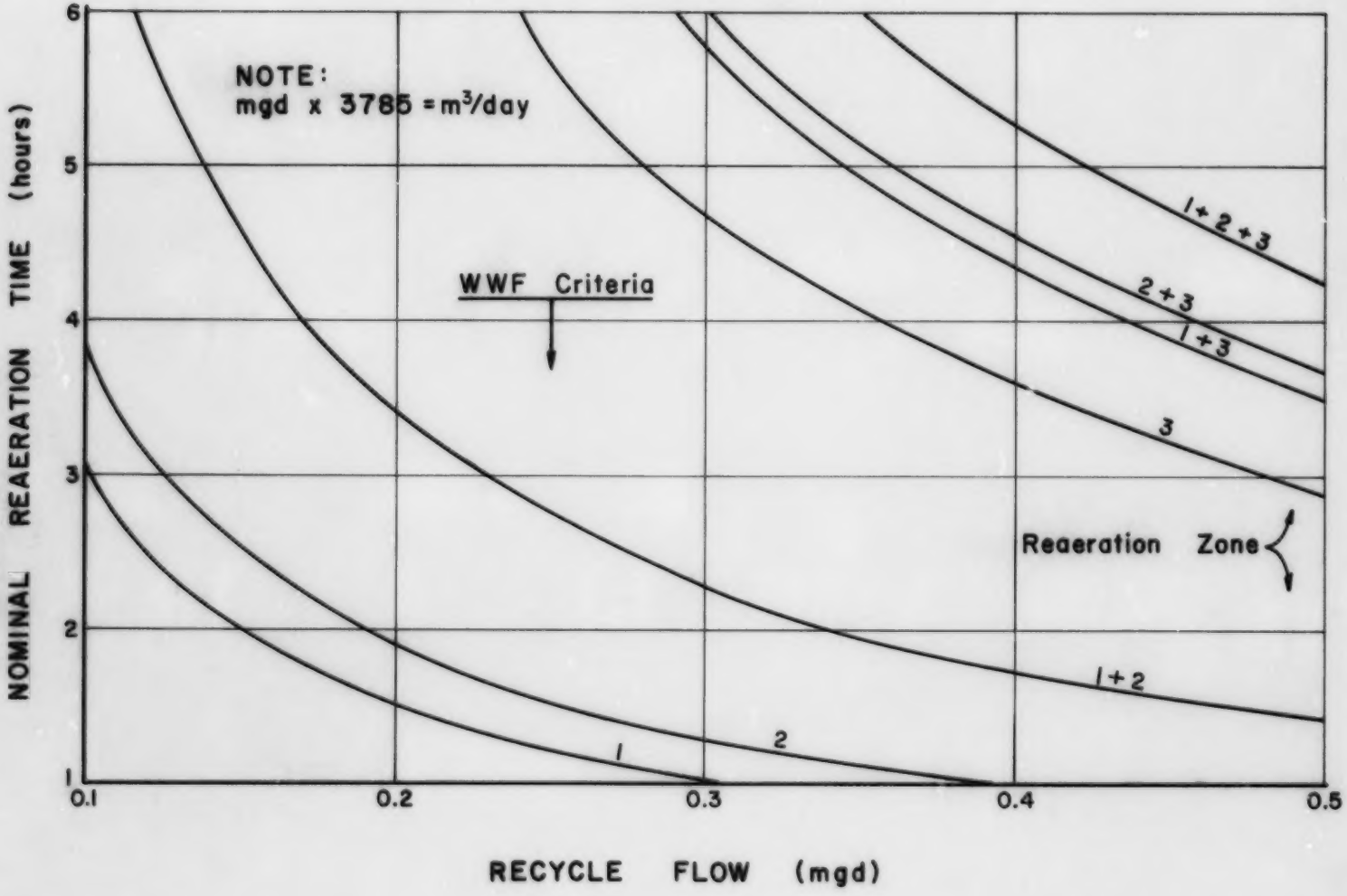

Figure 9: Reaeration Time Relationships

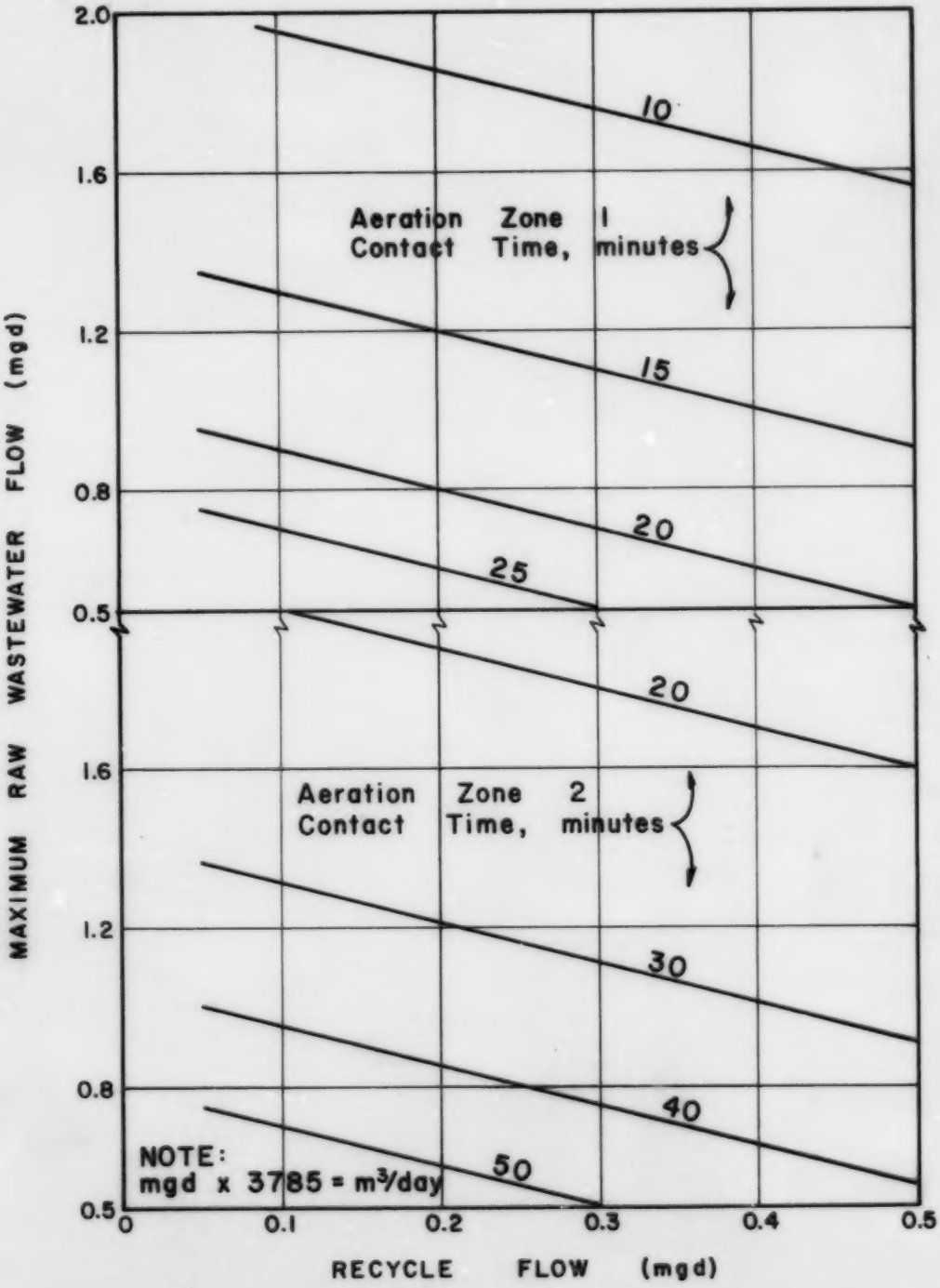

Figure 10: Raw Waste Flow Capacity For Various Contact Times

reaeration zone which is required to meet a given reaeration time for the range of recycle flows normally operable at Clatskanie (see also Table 2, Section 3) is defined. Thus, to meet the Kenosha WWF hydraulic criteria for reaeration time (1-4 hours) during the study, it can be seen that the contact stabilization system would need to be operated using reaeration Zones 1 and/or 2, unless recycle flows greater than 0.4 mgd (1514 m3/day) were required.* Other reaeration zones could be used to document effects with higher recycle flows and/or for longer reaeration times.

In Figure 10 the maximum wastewater flows which can be hydraulically processed with plant aeration/contact Zone 1 or 2 are shown as functions of recycle flow rate and nominal contact time. From this figure it can be seen that for recycle flows less than 0.5 mgd, either zone is capable of processing stormflows greater than 1.7 mgd (6434 m3/day) within the Kenosha nominal contact time criterion (greater than 10 minutes). To achieve more traditional contact times, either available flow capacity is reduced, or the process must be operated using both zones together to provide additional volumetric capacity.

Field surveys were designed such that for any particular sampling program the contact stabilization system was set up to operate at a pre-selected aeration/contact volume (Zone 1 or 2), and a preselected reaeration volume (Zone 1, 2 or 1+2). Selection of recycle flows and process volumes were based in part on criteria summarized in Figure 9 and 10, in part on biosolids criteria discussed subsequently, and in part on operating experience obtained at the plant on a continuing basis throughout the study. Insofar as possible within the scope of storm events experienced, these choices were made to document treatment capability with contact times and reaeration times at the low end of the scale of WWF criteria, since acceptable performance within this region would permit minimum process volumes in any future application.

5. Contact Stabilization - Biosolids Criteria: Several factors related to biosolids characteristics were considered in developing study scope. First, the interdependence of recycle flow rate, biosolids settleability and MLSS concentration held in the aeration/contact tank was recognized as an important study criterion. For an activated sludge system operating under ideal, steady-state conditions a balance must be achieved between these three variables. Mathematically this can be expressed by the relationship

$$(Q + R)(MLSS) = R(RAS) \qquad (1)$$

or

$$R/Q = (MLSS)/(RAS - MLSS) \qquad (2)$$

where, in consistent units,

Q = Wastewater flow rate,
R = Recycle flow rate,
MLSS = Contact tank biosolids concentration, and
RAS = Recycle biosolids concentration.

The RAS can be further defined in terms of the operating parameter SVI (sludge volume index) by use of the relationship

$$\text{RAS, mg/l} = 1{,}000{,}000/\text{SVI} \qquad (3)$$

Combining this relationship with Equation 2 then yields a formulation for R/Q in terms of both the mixed liquor solids concentration and the SVI:

$$R/Q = (\text{MLSS})(\text{SVI})/(1{,}000{,}000-(\text{SVI})(\text{MLSS})), \qquad (4)$$

where MLSS is in mg/l.

This ideal, steady-state formulation has often been used in practice to provide a basis for controlling activated sludge systems which are subjected to flow increases from storm events. Thus, it can be seen that if basic settleability characteristics do not change in response to the storm event (i.e., SVI remains constant) the MLSS level, and therefore treatment stability, can be maintained by varying R to keep a constant R/Q value. Although operational control of activated sludge systems based on this relationship has been widely employed and may be effective in many circumstances, recent studies (49) have indicated that secondary clarifier depth and biosolids thickening properties may also need to be considered to maintain process stability during transient, storm flow phenomena. As noted previously, recycle flows used during the Clatskanie study were selected in part on the basis of hydraulic criteria and in part on the basis of biosolids requirements discussed above.

Changes in mean cell retention time (MCRT) were also recognized as an important biosolids consideration in developing study scope. In the Kenosha, Wisconsin study (46) treatment effectiveness of the WWF contact stabilization system deteriorated if biosolids were held on a standby basis in a static state for a time period (stabilization time) which exceeded 5 days.* This stabilization time criterion for biosolids viability represents, in effect, a static MCRT for standby WWF biosolids beyond the operational MCRT of the parallel dry weather flow activated sludge facility from which biosolids were obtained for contact stabilization treatment of WWF. The study further noted that longer stabilization times could be effective with continuous application of waste activated sludge from the parallel DWF system to the standby contact stabilization system.

*See discussion under "Contact Stabilization Treatment", Section 5.

In an activated sludge system MCRT is related to the food-to-microorganism ratio, F:M. In practice, a wide range of MCRT values can be experienced (27, 50) for any given F:M ratio due to changes in waste water composition as well as to other factors. Thus, in planning the Clatskanie investigation the differences between the Clatskanie treatment facility and the system studied at Kenosha were considered to be important factors for defining biosolids viability requirements in the study. A special standby sludge study was developed as part of this investigation in order to define and assess viability criteria at Clatskanie. The scope of this study is discussed in a subsequent section of this report.

Storm Reconnaissance Procedures

Storm reconnaisance for the project consisted of three diffierent elements. First, regional weather forecasts were monitored to determine when a storm front was entering the area. Second, when a storm front was entering the area, the National Weather Service Office at Astoria, Oregon was called. An opinion was given on whether one inch or more of rain could be expected, and if more than one inch of rain was expected, the operators at the plant were alerted. This level of reconnaisance was not always the most reliable since the weather in the area was difficult to forecast. Although Astoria is only 40 miles west of Clatskanie, Astoria is on the Pacific Ocean whereas Clatskanie is amongst the coast range mountains just south of the Columbia River. Sometimes forecast storms would past over Clatskanie and other times transient storms would generate significant precipitation.

To augment the National Weather Service forecasts, the third level of reconnanisance, on-site monitoring was required. This was accomplished by the plant operators. If it started raining and plant flow reached 0.4 MGD (1514 m3/day), the process was changed and the samplers were started.

Data Collection Procedures

To define effluent control capabilities and process performance characteristics of the treatment facility, and to identify factors affecting process performance, a flexible WWF sampling program was developed which could meet the wide range of storm conditions anticipated at Clatskanie. In general, the intent of this WWF sampling program was to obtain time profiles of raw waste loads and process effluent as runoff from the storm event passed through the plant. In conjunction with this profile sampling, process monitoring was carried out to define changes in operational variables and parameters during storm events.

Time-profile monitoring was accomplished using three automatic samplers capable of discrete sampling on a short time sequence. Details of these samplers are given subsequently. The samplers were installed at three locations which were selected to obtain representative information on pretreated raw waste (after grit removal and comminution), on primary

clarified effluent, and on secondary process output prior to chlorination. Figure 11 shows one of the samplers installed on the secondary clarifier overflow. Additional samples were collected manually during various phases of the program to obtain data on the following:

- o Chlorinated Secondary Effluent;
- o Primary Settled Sludge;
- o Contact/Aeration Mixed Liquor;
- o Reaeration Biosolids;
- o Secondary Settled Biosolids; and
- o Standby Biosolids for Storm Events

Tables 4 and 5 summarize the range of samples obtained and analyses conducted in the Clatskanie investigation.

An integral part of the study design involved development of a data base for DWF conditions, from which performance comparisons for storm events could be made. Two approaches were used to develop this data base. First, compliance monitoring data were scanned to identify DWF regimes which were unaffected by storm events. These data were then analyzed statistically to provide a measure of long-term treatment capability during DWF conditions. A second approach involved an intensive sampling program over a 24-hour period during DWF conditions. This program was designed to obtain information on hourly variations at the plant under the DWF regime.

Details of the automatic sampling equipment, sampling procedures, analytical techniques, and plant flow measurement are presented in the following sections. In some cases outside laboratory services were used and these are discussed as well.

1. Sampling Techniques: Three automatic discrete samplers having 24 - 1000 ml sample bottles were used to sample the influent (preliminary treated), primary effluent and secondary effluent. In addition to having the capability of placing progressive samples in progressive bottles, the samplers also had the capability of filling several bottles in one sample event or using several sample events to fill one bottle.

Sample events could be actuated either on a time sequence basis or a flow proportional basis. Time sequence sampling was accomplished by a quartz crystal clock control in each sampler. These clocks were found to be very accurate and sampling frequencies could be selected ranging from 3.7 minutes to 24 hours. This mode of operation could be used in the sampling program when it was important that samples be taken at different times regardless of the flow passing through the plant.

Flow actuated sampling was accomplished by a signal integrater, which was connected to the effluent flow meter. The integrater would sense when a pre-set volume had passed through the plant and at that time it would activate all of the samplers. The volume required to activate the sampler

Figure 11: Sampler Installation on Secondary Clarifier Overflow

Table 4: Range of Samples Obtained at Clatskanie

Sampling Location	Type of Sampling	
	Automatic	Manual
Raw Waste, Pretreated [a]	X	
Primary Effluent	X	
Secondary Clarifier Overflow [b]	X	
Primary Settled Sludge		X
Secondary Effluent, Chlorinated		X
Mixed Liquor		X
Secondary Settled Solids		X
Stabilized Solids (Contact Mode)		X

[a] After grit removal and comminution
[b] Before chlorination

Table 5: Summary of Analyses Conducted at Clatskanie

Analysis/Measurement	Raw Waste	Prim. Effluent	Second'y Clarifier Overflow	Second'y Effluent, Chlorinated	Primary Sludge	Mixed Liquor	Second'y Sludge	Stabilized Sludge
Flow (and/or Volume)	X	X	X		(X)		X	
Total BOD5	X	X	X					
Dissolved BOD5	X	X	X					
Total Organic Carbon	X	X	X					
Chemical Oxygen Demand	X		X					
Total Solids					X		X	
Total Volatile Solids					X		X	
Total Suspended Solids	X	X	X			X	X	X
Total Vol. Susp. Solids	X	X	X			X	X	X
Kjeldahl Nitrogen	X	X	X		X	X	X	
Ammonia Nitrogen	X	X	X		X	X	X	
Oxidized Nitrogen [a]	X	X	X		X	X	X	
Total Phosphorus	X	X	X		X	X	X	
Dissolved Oxygen						X		X
pH	X	X	X		X	X	X	X
Temperature	X				X	X		X
Chloride	X	X	X					
Chlorine Residual				X				
Fecal Coliform			X	X				
Heavy Metals [b]	X	X	X		X	X	X	
Respiration Rate	X	X	X		X	X		X
Sludge Volume Index						X		X

[a] Oxidized Nitrogen = Nitrate plus Nitrite Nitrogen

[b] Heavy Metals Analyzed: Pb, Zn, Cd, Cu, Ni, Hg, Cr

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could be set over a wide range of values. A small strip chart recorder was used to record the times that the samples were taken.

Flow actuated sampling was used almost exclusively for storm sampling programs, since it has the advantage of concentrating sampling during times of high flows. In addition, sample compositing was considerably less complex, as discussed later in this section.

After a review of data gathered in initial storm surveys, it was discovered that analytical results from individual samples were highly variable, although general trends seemed reasonably consistent. To combat this variability a system was developed where eight small samples were taken at a high frequency (flow-proportioned) and deposited in each sample bottle. These samples were called "mini-composites". Use of these mini-composites was designed to reduce extreme variability resulting from automatic grab sampling.

Sampler intake velocity and the velocity of the liquid stream around the intake was considered. If velocities are too low solids have a tendency to settle out or travel in the lower portions of the stream. It has also been found that a large difference between the stream velocity around the sampler intake and the intake velocity of the sampler can cause samples to be less representative. The difference in velocities can result in segregation of particles drawn into the sampler, but if a difference in velocities must exist, it is better that the intake velocity exceeds the stream velocity (53). With these conditions in mind, wherever possible sample locations were selected which were directly downstream of a drop in the hydraulic grade line. This drop would result in mixing velocities in the stream, but the net velocity of the stream was not necessarily increased. According to manufacturer information the intake velocity for the samplers used was not less than 3 feet per second (91.4 cm/sec). Net stream velocities around sampler intakes with a 1.0 mgd (3785 m3/day) plant flow were calculated to be as follows:

- Influent — 2.1 fps (64.0 cm/sec)
- Primary Effluent — 1.6 fps (48.8 cm/sec)
- Secondary Effluent — 0.65 fps (19.8 cm/sec)

Flow proportional composite samples were generally made at the end of sampling programs. If samples were gathered with the automatic samplers in a flow actuated mode, the composite was made by taking equal volumes from each bottle. If automatic samplers were in a time actuated mode, the volume of flow was determined which passed through the plant during each sample period. The volume which was composited from samples were then proprotional to this volume of flow.

Manual sampling was employed primarily for process monitoring. As in the case of automatic sampling, sample locations were selected where

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there was some turbulence and care was taken to insure that the sample was representative.

Sample locations for the various samples were as follows:

- o Primary Settled Sludge -- Sample was taken from the launder near the primary sludge pump discharge.

- o Secondary Effluent, Chlorinated -- Sample was taken near the chlorine contact chamber discharge weir. If the sample was to be used for fecal coliform analysis, a sterile sample container was used.

- o Mixed Liquor -- Sample was taken directly from the aeration basin.

- o Secondary Settled Solids -- Sample was taken from the launder near the secondary solids return pump. This stream was alternately channeled into the reaeration basins, or digester for wasting.

- o Stabilized Solids -- Sample was taken directly from the reaeration basin or basins. If two basins were in use, a sample was taken from each.

Early in the demonstration study, attempts were made to determine the actual hydraulic retention time of the plant. With this information a slug load could have been followed through the plant and treatment accomplished on this slug could have been more thoroughly documented. Results of the hydraulic retention time studies were inconclusive and only one phenonena was detected: the actual retention time appeared to be significantly less than the nominal retention time. The actual retention times observed varied considerably, and no conclusions could be drawn. It was speculated that the high recycle rates used both in the primary DAF and the secondary process could have created a nearly complete mix condition, thereby obscuring the effect of the hydraulic retention time. Another factor, considered along with the effect of hydraulic retention time, was the effect of increased flow. If the influent flow increased and a slug of high strength sewage arrived simultaneously, the increased flow would have an immediate effect on the plant effluent while the slug of high strength sewage would not have an effect until later. After considering these points it was decided that the effect of slug flow should be dealt with in the data analysis plase. To account for this concern more emphasis was placed on mass rates rather than concentrations, and wherever possible a longer time base was used to evaluate the data.

2. Analytical Techniques: Analyses performed can generally be divided into two groups; those used for time profile monitoring and those used for process monitoring. Time profile analyses consisted mainly of total BOD and TSS. These analyses were generally performed on each "mini-composite" sample, as well as on composite samples over longer

periods of time. Total volatile suspended solids (TVSS) analyses were performed on some samples. Analyses were all performed according to Standard Methods for the Examination of Water and Wastewater (51). A millipore filter assembly with 4.25 cm filters was used for TSS and TVSS analyses and a dissolved oxygen meter was used to measure dissolved oxygen levels in the BOD bottles.

Process monitoring consisted of mixed liquor suspended solids (MLSS), mixed liquor volatile suspended solids (MLVSS), total solids (TS), total volatile solids (TVS), sludge volume index (SVI), pH, respiration rate and centrifuge suspended solids. All of these analyses except respiration rate and centrifuge suspended solids analyses were performed as described in Standard Methods. MLSS and MLVSS analyses utilized 9 cm filter discs. The respiration rate analysis and the centrifuge suspended solids analysis are not listed in Standard Methods. A brief description of these analyses follows. A more detailed procedure is included in Appendix A.

- Respiration Rate Analyses -- A sample of mixed liquor from either the aeration basin or the stabilization basin was gathered. As soon after the sampling as possible, the sample was saturated with DO by shaking the sample in a half-filled bottle. The oxygen uptake rate was then measured by using the DO meter to measure the DO concentration drop over a ten minute period. The respiration rate was then determined by multiplying the DO drop by 6,000 and dividing by the MLSS of the sample. The units for respiration rate are mg-02/hr/gm-SS.

- Centrifuge Suspended Solids Analysis -- This analysis was used occasionally during the study as a quick method of determining the MLSS concentration. A sample of mixed liquor was poured into a calibrated cone-shaped centrifuge tube. This tube was then centrifuged for 10 minutes at approximately 4750 rpm. After centrifuging the level of solids in the tube was read and recorded. The estimated MLSS of the sample was then calculated by using a ratio number. This ratio number was developed by running parallel centrifuge and MLSS analyses on a similar sample.

Additional process monitoring analyses were performed in situ. These included dissolved oxygen concentration (DO), temperature, and sludge blanket level (SBL). The DO and temperature were determined using a portable DO meter. The SBL was determined by slowly lowering a transparent 1" diameter plastic pipe to the bottom of the clarifier in a vertical position. The tube had graduations on the outside and a foot valve on the bottom so that when the tube was removed from the clarifier a complete cross section of the clarifier contents could be observed. The sludge blanket level was then recorded in feet and represents the depth of the sludge blanket on the bottom of the clarifier. Values for SBL were used to calculate biosolids quantities in the secondary clarifier from which total system activated sludge solids (TASS) could be developed.

Outside laboratories were employed for some of the more complex and less routine types of analyses. These analyses included:

- o Demand Analyses

  Total Organic Carbon (TOC)
  Chemical Oxygen Demand (COD)
  Dissolved BOD
  Chlorides

- o Nutrient Analyses

  Ammonia Nitrogen
  Nitrate Nitrogen
  Nitrite Nitrogen
  Organic Nitrogen
  Total Phosphorus

- o Heavy Metal Analyses

Lead	Nickel
Chromium	Cadmium
Copper	Mercury
Zinc	

All of these analyses were performed as described in Standard Methods (51) except dissolved BOD which is not listed. The dissolved BOD analyses were performed by first filtering the sample through a 0.45 micron filter and then performing the standard BOD analysis with seed. For TOC analyses a Beckman direct injection analyzer was used.

3. Flow Measurement: Measurement of the main flow through the plant was accomplished at three locations: (1) The influent pipelines, (2) the primary influent pipeline, following preliminary treatment, and (3) the final effluent.

Measurements at the influent pipelines were accomplished by two 6-inch magnetic flow meters on each of the influent pipes (see Figure 3). These meters were connected to recorders and flow totalizers. The meters were fitted with ultrasonic cleaners which were operated periodically. According to the manufacturers specifications these meters are accurate to plus or minus 0.02 mgd (76 m3/day). This was the most accurate of the flow measuring points and thus the influent flow readings were used to document plant flow characteristics.

The primary influent flow was measured by a 6-inch magnetic flow meter. This flow meter was connected to a strip chart recorder and a flow control valve. The main function of the primary influent meter was to limit the maximum flow through the primary clarifier.

The final effluent flow was measured by a 6-inch Parshall flume with a level recording meter. The meter was connected to a strip chart recorder, a flow totalizer, a chlorinator control, and the sampler integrater control.

Flow measurement of side streams was accomplished in four locations at the plant. These included: (1) the dissolved air flotation recycle stream, (2) the return activated sludge stream, (3) the stream entering the stabilization basins, and (4) the waste stream to aerobic digestion.

The dissolved air flotation recycle measurement was accomplished by an orifice plate with a differential pressure meter. The remaining installations were all Palmer-Bolis flumes with level recording meters.

Statistical Analyses

A computer-based statistical package was developed for analyzing time-series information obtained from the field data collection program. Series were obtained on a concentration basis as well as on a mass rate basis. Output from this package yielded the arithmetic mean, minimum and maximum values for each series along with standard deviation and skewness values. Probabilities of occurrence were also defined at the 10, 50 and 90 per cent levels. Process TSS and BOD removals were calculated using arithmatic mean values from the analyses on a mass rate basis.

Process operational parameters were calculated for each time-series and then analyzed using the above statistical package. Parameters calculated included primary and secondary clarifier nominal hydraulic retention times and overflow rates based on waste flow (without recycle), and nominal aeration/contact time, both with and without recycle.

SECTION 7

RESULTS OF STUDY

During the two-year period of investigation, nine storm events and two DWF regimes were sampled. Several of the storm events yielded long-term flow patterns (see Section 6) which afforded the opportunity to vary study conditions and/or analyze distinct sequences within a single event. Table 6 summarizes the scope of the surveys conducted in terms of process configurations used, flow regimes observed, and the nature of data collected. Analysis and evaluation of the results obtained are presented subsequently in this Section.

Rainfall-Runoff Characterization

To understand the relationship between rainfall and sewer flows at Clatskanie it is necessary to consider the I/I sources in the system. The principal source of I/I in the Clatskanie system is infiltration through leaky pipe joints. There are very few roof drains and no known catch basins connected to the system.

Because of this condition the storm runoff needs to percolate through the soil before it can enter the sewer. This has two effects on the relationship between rainfall and sewer flow. First it is necessary that the soil be saturated before sewer flows can increase. Second, the lag between the start of a rain storm and the subsequent increase in flow is greater than that of a system where there are catch basins connected.

If the soil was not saturated then the rainfall would be absorbed, and the sewer flow would not increase. This was particularly evident in the early fall when small storms would produce more than 1 inch of precipitation and there would be no noticeable effect on sewer flows. The same storm if it would have occured later in the year, after the soil was saturated would have resulted in a noteworthy flow, perhaps in the range of 0.5 to 0.7 mgd.

The lag between the time that the rain would begin and the sewer flow would increase was approximately one hour. This is longer than could be expected if the system had catch basins and direct inflow sources. The City of Rainier, Oregon is about 15 miles east of Clatskanie and it has a traditional combined sewer system with catch basins connected. The City

Table 6: Summary of Study Surveys

Survey No.	Date(s)	Flow Regime	Primary Mode	Secondary Configuration			Nature of Data Obtained
				Mode**	Aeration	Reaeration	
1	10/23-24/78	DWF	DAF	-	-	-	TSS removal across primary DAF under DWF conditions (Baseline).
2	11/30/78	WWF	DAF	CM	1+2	-	Overall plant TSS and BOD removal
3A	2/6-12/79*	WWF	Sed.	CM	1+2	-	Complete TSS and BOD performance data; TOC, heavy metal and nutrient data.
3B	2/6-12/79*	WWF	DAF	CM	1+2	-	Inconclusive due to power failure
3C	2/6-12/79*	WWF	DAF	CM	1+2	-	TSS removal across primary DAF
3D	2/6-12/79*	WWF	Sed.	CM	1+2	-	TSS removal across primary sedimentation
3E	2/6-12/79*	WWF	DAF	CM	1+2	-	TSS removal across primary DAF without pre-draining of settled sludge.
4	2/17-18/79	WWF	DAF	CM	1+2	-	TSS removal across primary DAF
5	3/19-20/79	DWF	Sed	CM	1+2	-	Intensive baseline survey for DWF, Complete TSS and BOD performance data. Heavy metal, nutrient and TOC data on processes. Heavy metal, nutrient, solids data on sludges.
6	10/18/79	WWF	DAF	CS	2	1	Complete TSS and BOD performance data; COD data
7	11/22/79	WWF	DAF	CS	2	1	Complete TSS and BOD performance data. Effect of high skimmer speed on primary DAF performance.

Table 6: Summary of Study Surveys (cont.)

Survey No.	Date(s)	Flow Regime	Primary Mode	Secondary Configuration Mode**	Aeration	Reaeration	Nature of Data Obtained
8	12/1-2/79	WWF	Bypassed	CS	2	1	TSS and BOD removal across plant without primary clarification. Primary unit was inoperable for this event.
9	12/14/79	WWF	DAF	CS	1	1+2	Complete TSS and BOD removal data
10A, 10B	12/17-19/79	WWF	DAF	CS	1	1+2	Complete TSS and BOD removal data; VSS data
11A, 11B	1/11-15/80	WWF	DAF	CS	1	1+2	Complete TSS and BOD removal data; COD data
12	10/4-11/30/79	DWF	Sed.	CM	1+2	-	Complete TSS and BOD removal data

* Letters indicate study conditions were varied and/or distinct sequences within the event were analyzed.

**CM = Complete-mix; CS = Contact Stabilization

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is very similar to Clatskanie in terms of size and topography. In general terms, the time lag experienced at Rainier is 20 to 30 minutes. This is similar to the calculated "time of concentration" for Clatskanie if the system had catch basins.

A third effect of the infiltration problem which is not necessarily related to rainfall is the relationship of ground water to river elevations. Many of the older sewers are located in bottom lands near the river and there is evidence that tides and river elevations have a low to moderate effect on sewer flow. During storms when river elevations were monitored it appeared that flow variations in the magnitude of 0.1 mgd could be attributed to tides and river elevations. Another possible source of this flow may have been tide gates on sewer bypasses which occasionally stuck partially open.

Attempts were made to relate cummulative rainfall amounts and flows for the Clatskanie system. It was found that there were considerable deviate values. This deviation was evidently the result of the need for saturated soil discussed earlier and the intensity of rainfall. Normally, the more intense the rainfall, the greater the degree of runoff. The greater degree of runoff would result in a smaller fraction of the rainfall percolating into the soil and finding its way into the sewers. The following table summarizes the cummulative rainfall required for various flows for some of the storms sampled. Rainfall intensities encountered over a one hour period ranged from 0.06 to 0.42 inches per hour.

Plant Flow (mgd)	Time Required From Rainfall Start (hr)	Cumulative Rainfall Required (inches)
0.5	3-13	0.20 to 0.44
0.7	4-28	0.40 to 0.96
1.0	8-15	0.67 to 2.00
1.5	18*	2.44*
2.0	20*	3.94*

inches x 2.54 = cm
mgd x 3785 = m3/day

*Limited Data

Flush Phenomenon -- Due to the flow characteristics of gravity sewers, as flows subside and stream velocities drop below a critical value, solids settle out of the stream and are deposited in the sewer line. The critical velocity depends upon the nature of the solids being

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deposited. The critical velocity for light organic matter is obviously much less than that of inorganic sand and grit particles. After solids have been deposited at low rates of flow these same solids are liable to be picked up again by the flow as velocities exceed the critical value. This process of picking up deposited solids by increased flow rates is often referred to as a "flush".

There are many time scales over which solids can be deposited and flushed. Plant operators often speak of the daily flush. This flush occurs when solids deposited during low early morning flows are picked up as flow increases at the breakfast hour. When a system is subjected to I/I flows this flush phenomenon can occur over the period between storms during the wet season. In this study this was referred to as the storm to storm flush. Another I/I related flush can occur over the seasonal period. This was called the seasonal flush and involved solids which were deposited during dry months and flushed out during the wet season. At Clatskanie all of these flush phenomena were observed and the later two I/I related flush phenomena were of importance in storm monitoring efforts.

The storm to storm flush was characterized as an organic flush. This is evident by analyses of samples taken during the flush. The BOD of the samples was nearly equal to the TSS and the suspended solids were nearly 100% volatile. This flush phenomena is further evidenced by the low BOD strength of the sewage during dry weather flow following a storm.

The storm to storm flush occurred at flows ranging from 0.4 to 0.6 mgd (1500 to 2300 m3/day). This was often the first flush that could be identified in the course of a storm. Peak TSS and BOD concentrations ranged from 230 to 386 mg/l for TSS and from 200 to 296 mg/l for BOD. More than one week of dry weather flow was required between storms in order to produce a storm to storm flush.

The seasonal flush consisted mainly of inorganics, this was typified by the low BOD values in relation to TSS and the low percentage of volatile suspended solids. These solids appeared to be sand and grit particles as well as finer clays. These solids found their way into the sewer over the year and required a high flow rate to carry them through. More of these solids were carried out as successively higher flows were reached during the wet season.

In the storms analyzed it appeared that generally the organic solids were flushed early in the storm. As the storm progressed and the flow increased the organic solids fraction diminished and the inorganic fraction increased. In the course of the storms it was often the case that as the rainfall intensity varied, the flow rate at the plant would increase to a level and then remain at that level for a period of time. Sometimes this steady state flow would continue for hours before it would either increase or diminish. The higher BOD and TSS concentrations which indicated the flush usually occurred immediately as the plant flow rate

increased to a new level as the flow continued at the same rate the BOD and TSS concentrations would decline.

Early in the wet season it was found that a noteworthy inorganic flush could be generated by a flow of 0.7 mgd (2600 m3/day), but as the wet season progressed and more of the seasonal solids were carried away, increasingly higher flows were required to generate a comparable flush.

Baseline Characterization of DWF

Four series of monitoring data at Clatskanie were used in developing a baseline characterization of DWF. Three of these series characterized representative performance with primary sedimentation and complete-mix activated sludge using aeration zones 1+2. The fourth was designed to document baseline DWF performance (low overflow rates) of the primary DAF process. The scope of each series is given below:

1. Routine Compliance Monitoring, 1978: Available operating records for DWF periods during 1978 were scanned to obtain 21 days of influent and effluent BOD data and 49 days of influent and effluent TSS data for time-series analysis. Only data for raw waste flows less than 0.3 mgd (1136 m3/day) were used in the analysis. BOD and TSS analyses for these data were on grab samples.

2. R & D Monitoring, 1979/80: During the 1979/80 survey period plant monitoring was maintained between storm events using automatic samplers to collect 24-hour composite samples. One DWF series was obtained from this sequence with 10 days of BOD and TSS data (see Table 6, Survey No. 12). Process operational data were also obtained in this series.*

3. Intensive Sampling Survey: One intensive survey (see Table 6, Survey No. 5) was made over a 24-hour period of operation during DWF conditions. Two-hour composites were collected for TSS and BOD performance analysis over the 24-hour survey period. Process operational data and information on heavy metals, nutrients, TOC and sludge characteristics were also collected. Results for these latter four types of data are presented subsequently in this Chapter under the title, "Supplementary Studies".

4. DAF Sampling Survey: One survey (see Table 6, Survey No. 1) was made to document DAF performance at low overflow rates. Grab samples were collected from the DAF clarifier over a two day period between the hours of 8 am and 5 pm. Samples were analyzed for TSS only. Routine plant operational data were used to define process operating conditions.

Table 7 summarizes overall plant performance characteristics under DWF conditions for the three data series obtained with primary sedimentation. More complete variability characteristics for these series

Table 7: Summary of Plant Performance under DWF Conditions*

Plant Characteristic	Results of Three Series: Range	Overall Average
Flow (mgd)		
Mean	0.18-0.21	0.19
Maximum	0.23-0.28	0.25
Raw Waste TSS (mg/l)		
Mean	145-198	166
Maximum	267-468	359
Raw Waste BOD (mg/l)		
Mean	186-234	208
Maximum	264-387	324
Primary Effluent TSS (mg/l)**		
Mean	54-56	55
Maximum	98-102	100
Primary Effluent BOD (mg/l)**		
Mean	98-124	111
Maximum	162-168	165
Secondary Effluent TSS (mg/l)		
Mean	6-14	10
Maximum	14-25	19
Secondary Effluent BOD (mg/l)		
Mean	8-19	13
Maximum	15-26	22
TSS Removals (%)		
Primary	66-73	70
Secondary	73-81	77
Plant	91-96	94
BOD Removals (%)		
Primary	39-57	48
Secondary	86-88	87
Plant	91-96	94

*More complete data for each series is provided in Appendix B.
**Data available for 1979 Monitoring Series and Intensive Survey only.

mgd x 3785 = m3/day

are given in Appendix B. Table 8 gives representative operating conditions for the three series based on mean values. Ranges are also summarized in Appendix B.

Based on results from the three series it can be seen that representative DWF raw waste TSS concentrations are between 145 and 198 mg/l (mean = 166 mg/l), with peaks as high as 468 mg/l. DWF BOD concentrations are typically between 186 and 234 mg/l (mean = 208 mg/l), with peaks up to 387 mg/l. These mean concentrations translate to raw waste mass loadings for TSS and BOD of 263 and 330 lb/day (119 and 150 kg/day), respectively, based on an average dry weather flow value of 0.19 mgd (719 m3/day).

DWF removals of TSS and BOD by primary sedimentation for the three data series were 66-73 and 39-57 percent, respectively, with transient values as low as 36 and 15 percent. Mean primary overflow rates for these performance data varied between 575 and 670 gpd/sq ft (236 and 275 m3/m2/day).

Overall plant removals for both TSS and BOD were 91-96 percent, while removals for the secondary process alone were 73-81 percent for TSS and 86-88 percent for BOD. Mean effluent TSS and BOD concentrations of 6-14 mg/l and 8-19 mg/l were observed, with maximum values of 25 and 26 mg/l, respectively.

During the DWF studies, secondary process F:M values, MCRT values, and clarifier overflow rates were typically less than 0.14 kg-BOD/day/kg-MLSS, 7 days, and 130 gpd/sq ft (5 m3/m2/day), respectively, based on arithmetic means. Mean biosolids respiration rates and MLSS levels varied between 5.0 and 11.9 mg-02/hr/gm-SS, and 3930 and 7560 mg/l, respectively.

Table 9 summarizes baseline DWF performance and operating data for DAF primary clarification. It can be seen that a mean TSS removal of 56 percent was achieved, with transient values between 72 and 10 percent. The process was subjected to raw waste TSS levels between 113 and 198 mg/l (mean = 158 mg/l) during this survey, with flow varying between 0.15 and 0.21 mgd (568 and 795 m3/day). These flows yielded overflow rates, based on raw waste flow alone, between 480 and 670 gpd/sq ft (20 and 28 m3/m2/day) with a mean of 570 gpd/sq ft (23 m3/m2/day). For this survey the DAF process was operated at a mean recycle flow of 120 gpm (20.9 m3/day), a mean air rate of 49 scfh (1.37 m3/hr), and a mean air pressure of 68 psig (469 kN/m2).

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* Other data collected during interim monitoring yielded results with flows which were greater than 0.3 mgd (1136 m3/day), which are outside of the DWF regime.

Table 8: Operating Conditions for DWF Baseline

Operating Condition	1978	1979	No.5
MLSS, mg/l	3930	4690	7560
SRS, mg/l	-	10025	11800
MLSS, lb	1429	1144	2700
TASS, lb	-	1830	-
Temp. C	17.5	15.0	14.0
Resp. Rate, mg-02/hr/gm-SS	13.0	11.9	5.0
SVI	165	171	118
SBL, ft	-	1.25	-
WAS, lb/day	-	254	394
F:M (MLSS-basis)	-	0.14	0.09
F:M (TASS-basis)	-	0.08	-
MCRT (MLSS-basis), days	-	4.4	6.9
MCRT (TASS-basis), days	-	7.6	-
Prim. Over. Rate, gpd/sq ft	575	600	600
Sec'y Over. Rate, gpd/sq ft*	115	120	130

lb x 0.4536 = kg
ft x 30.48 = cm
gpd/sq ft x 0.041 = m3/m2/day
*Based on raw waste flow.

Table 9: Baseline DWF Data for DAF Primary Clarification

Characteristic	Arith. Mean	Maximum	Minimum	Stnd. Dev.	Skew-ness
Flow, mgd	0.17	0.21	0.15	0.02	0.06
Raw Waste TSS, mg/l	158	198	113	30	0.14
DAF Effluent TSS, mg/l	70	115	48	20	1.12
TSS Removal, %	56	-	-	-	-
Recycle Flow, gpm	120	140	100	-	-
Air Pressure, psi	68	70	65	-	-
Air Rate, scfh	49	68	30	-	-
Overflow Rate, gpd/sq ft*	570	670	480	70	0.05

*Based on raw waste flow; gpd/sq ft x 0.041 = m3/m2/day
mgd x 3785 = m3/day
psi x 6.89 = kN/m2
gpm x 0.091 = m3/day
scfh x 0.028 = m3/hr

WWF Characterization

Of the eighteen field surveys conducted during the investigation (see Table 6), fifteen were carried out under WWF conditions. A summary of raw waste concentration and mass rate characteristics for these surveys is given in Tables 10 and 11.

Mean wet weather flows experienced during the study varied from 0.47 to 1.61 mgd (1779-6094 m3/day), with peak values ranging from 0.53 to 2.31 mgd (2006-8743 m3/day). The ratio of peak flow to mean flow varied between 1.04 and 1.65. Variability patterns were also varied, as evidenced by the range of standard deviations and skewness values provided in Table 10.

Like flow rate, raw waste TSS and BOD concentrations measured during WWF periods also varied widely. Mean TSS values ranged between 79 and 179 mg/l, with peak concentrations as high as 455 mg/l. Ratios of maximum TSS to mean TSS were between 1.34 and 3.42.

Table 10: Summary of WWF Concentration Characteristics

Survey No.	Flow (mgd)				BOD Concentration (mg/l)				TSS Concentration (mg/l)				
	Mean	Max.	Stnd. Dev.	Skew-ness	Mean	Max.	Stnd. Dev.	Skew-ness	Mean	Max.	Stnd. Dev.	Skew-ness	TSS/BOD*
2	0.95	1.29	0.21	0.27	87	200	44	1.63	146	455	112	1.99	1.68
3A	0.65	0.84	0.17	-0.18	71	115	39	0.14	161	294	112	0.26	2.27
3B	1.45	1.64	0.20	-0.67	25	35	9	0.26	165	347	114	0.63	6.60
3C	0.57	0.72	0.09	0.14	-	-	-	-	76	160	36	0.91	-
3D	0.51	0.59	0.05	0.00	-	-	-	-	169	282	60	0.58	-
3E	0.63	0.72	0.07	0.15	-	-	-	-	108	145	25	0.60	-
4	0.71	0.85	0.06	1.38	-	-	-	-	143	301	84	0.56	-
6	0.47	0.53	0.06	-0.52	243	396	79	1.06	179	386	114	0.71	0.74
7	0.61	0.72	0.08	0.00	135	200	43	0.53	161	234	64	-0.06	1.19
8	0.90	1.22	0.13	1.14	37	78	20	0.13	115	170	37	-0.18	3.11
9	0.54	0.65	0.09	-0.17	61	120	36	0.25	88	160	47	0.19	1.44
10A	1.07	1.25	0.07	1.06	59	114	25	1.26	112	226	64	0.76	1.90
10B	0.75	0.86	0.07	-0.34	39	54	13	-0.38	153	278	64	0.26	3.92
11A	1.61	2.31	0.53	-0.68	50	129	27	1.00	91	216	50	0.83	1.82
11B	1.02	1.68	0.29	0.87	35	66	14	0.80	79	118	21	0.40	2.26

mgd x 3785 = m3/day

*Based on mean values

Table 11: Summary of WWF Mass Rate Characteristics

Survey No.	BOD Mass Rate (lb/d)				TSS Mass Rate (lb/d)			
	Mean	Max.	Stnd. Dev.	Skew-ness	Mean	Max.	Stnd. Dev.	Skew-ness
2	674	1151	295	0.46	1098	2618	648	1.19
3A	422	748	285	0.02	959	1765	757	0.17
3B	422	811	280	0.48	1997	4341	1432	0.72
3C	-	-	-	-	352	627	140	0.29
3D	-	-	-	-	738	1388	317	0.82
3E	-	-	-	-	562	677	86	0.36
4	-	-	-	-	876	2134	597	0.93
6	966	1684	378	1.00	731	1642	500	0.72
7	702	1134	258	0.55	836	1270	361	-0.09
8	273	572	141	0.16	893	1730	410	0.65
9	292	580	189	0.13	425	841	275	0.22
10A	520	960	200	1.20	994	1904	546	0.73
10B	252	387	93	-0.26	956	1808	389	0.44
11A	586	1040	221	0.69	1199	3622	859	1.30
11B	317	925	218	1.68	680	1329	306	0.74

lb/d x 0.4536 = kg/day

Mean BOD concentrations measured during the survey varied in magnitude between 25 and 243 mg/l. Peak BOD concentrations reached 396 mg/l, with concentration ratios (maximum/mean) between 1.38 and 2.58.

Mass rate calculations yielded mean raw waste TSS and BOD loads between 352 and 1997 lb/day (160-906 kg/day), and 252 and 966 lb/day (114 and 438 kg/day, respectively. Maximum loads were 627-4341 lb/day (284-1969 kg/day) for TSS and 387-1684 lb/day (176-764 kg/day) for BOD. Ratios of maximum load to mean load varied between 1.20 and 3.01 for TSS and between 1.52 and 2.92 for BOD.

Figure 12 shows a plot of mean flowrate for the WWF surveys identified in Table 6 versus corresponding mean raw waste TSS and BOD concentrations. Data for DWF series are also included. The figure shows that as mean flow rate increased for the surveys made there was generally a decrease in both raw waste TSS and BOD concentration applied to the plant. This trend was most pronounced for BOD at mean flows greater than about 0.5 mgd (1892 m3/day).

For any given storm event and/or survey sequence sampled during the investigation, time-concentration profiles were varied. Figures 13 illustrates two patterns observed.

A plot of mean flow versus mean raw waste TSS/BOD ratio for both WWF and DWF is shown in Figure 14. The plot suggests an increasing trend up to about 1.1 mgd (4164 m3/day). Data is limited beyond this point, but the one survey point shown suggests that a leveling occured at higher flow regimes sampled. Wide variations in hourly TSS/BOD ratios would be expected during progress of any individual storm event.

Effluent Control and Plant Performance

Eight surveys were made under WWF conditions using the contact stabilization mode of operation. All surveys except one were conducted using primary DAF. One was conducted in the absence of any primary clarification. A summary of flow, input loads applied to the secondary process, and effluent quality for each of these surveys is given in Table 12. Process operational factors are summarized in Table 13. Variability characteristics for each survey are given in Appendix B.

For each of the surveys identified in these tabulations effluent quality was compared and contrasted to design/performance criteria for WWF. This comparison is shown in Table 14. Where survey conditions were such that plant operation was outside of the range of criteria given, an "X" is indicated in the tabulation.

Inspection of Table 14 shows several relationships. First, for four surveys having mean flows between 0.47 and 1.07 mgd (1779 and 4050 m3/day), and for which plant operation was within all WWF criteria, mean effluent TSS concentrations were between 2 and 24 mg/l, while mean

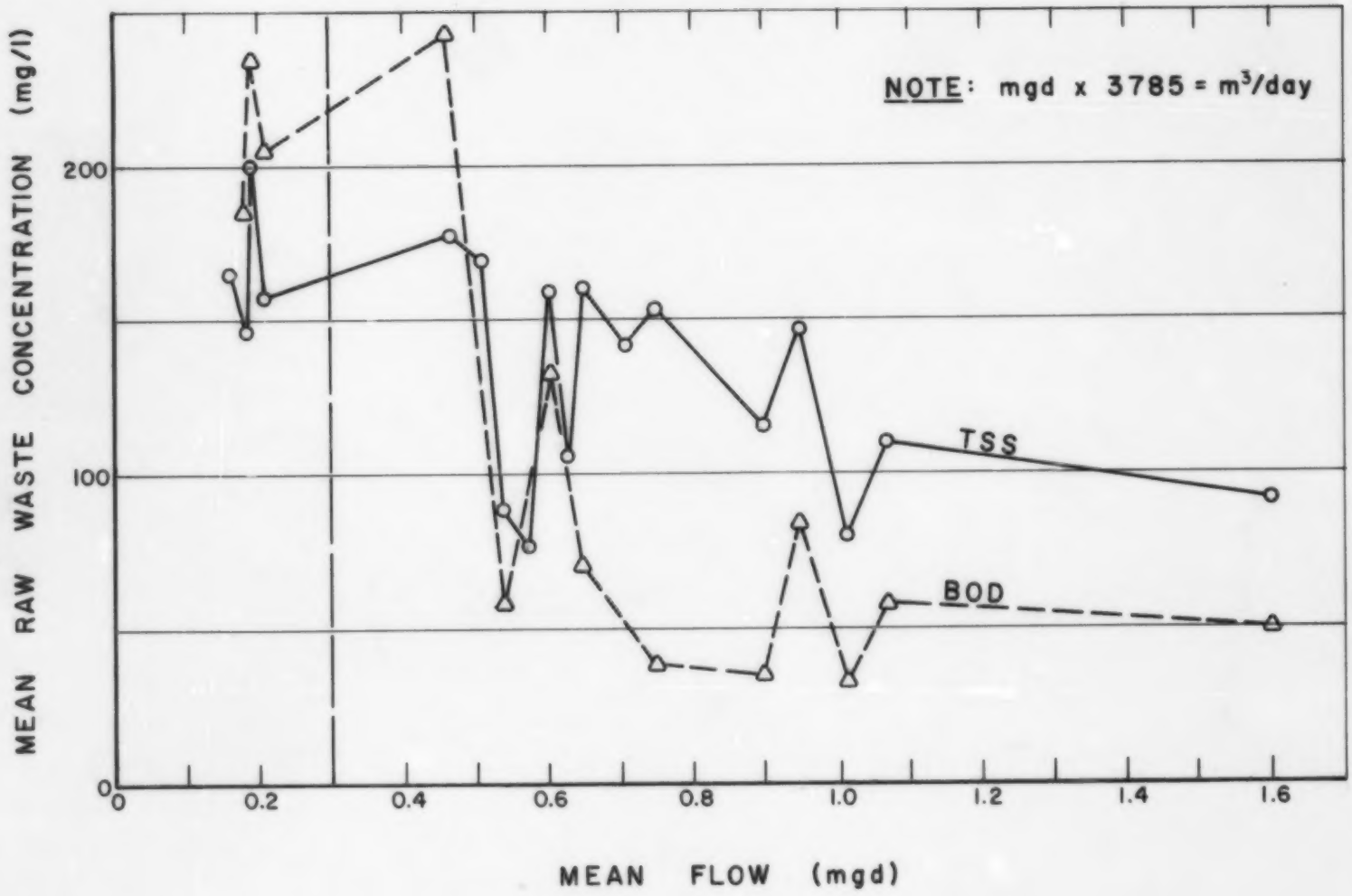

Figure 12: Raw Waste Concentration vs. Flow

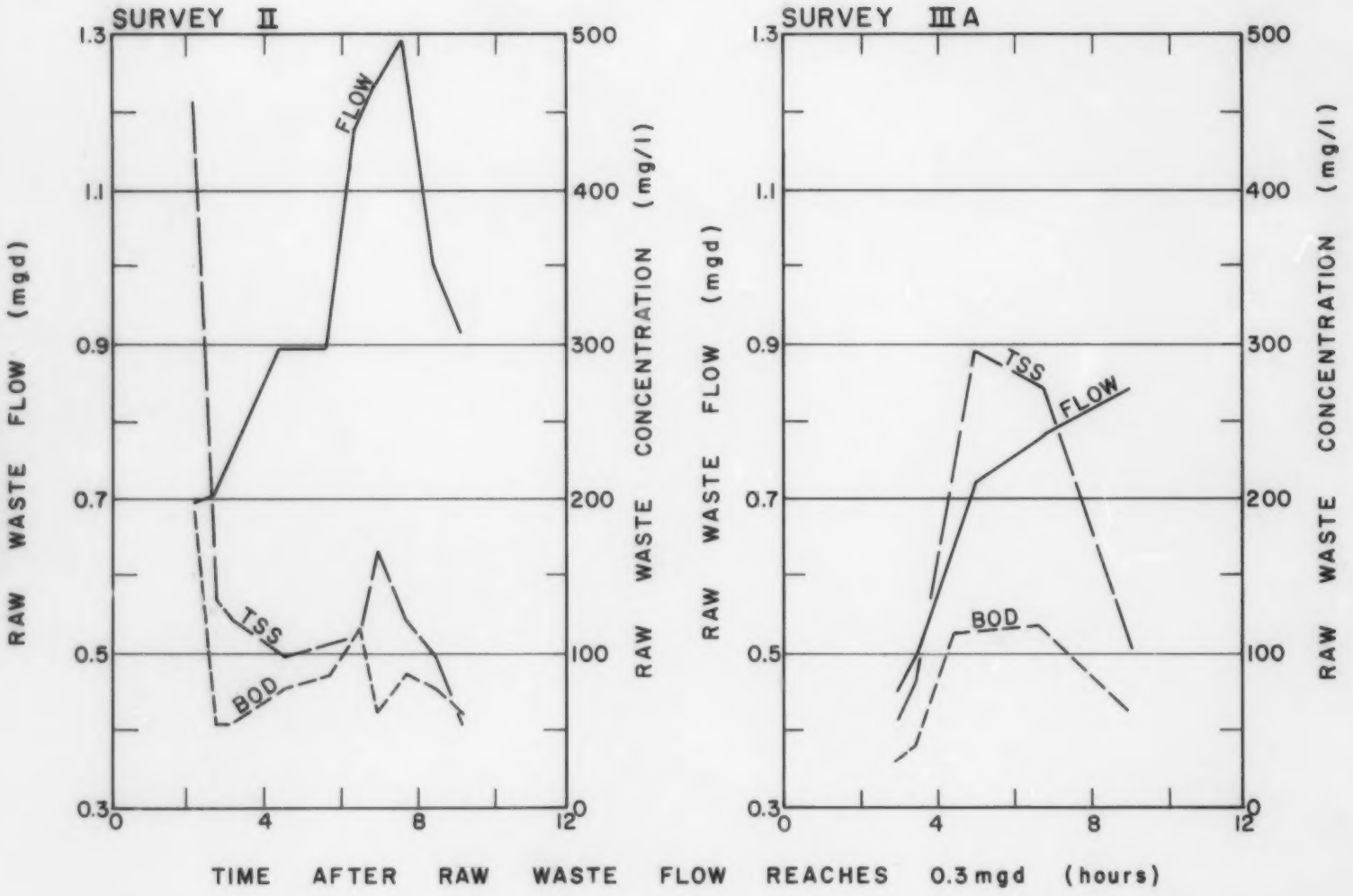

Figure 13: Time Concentration Profiles

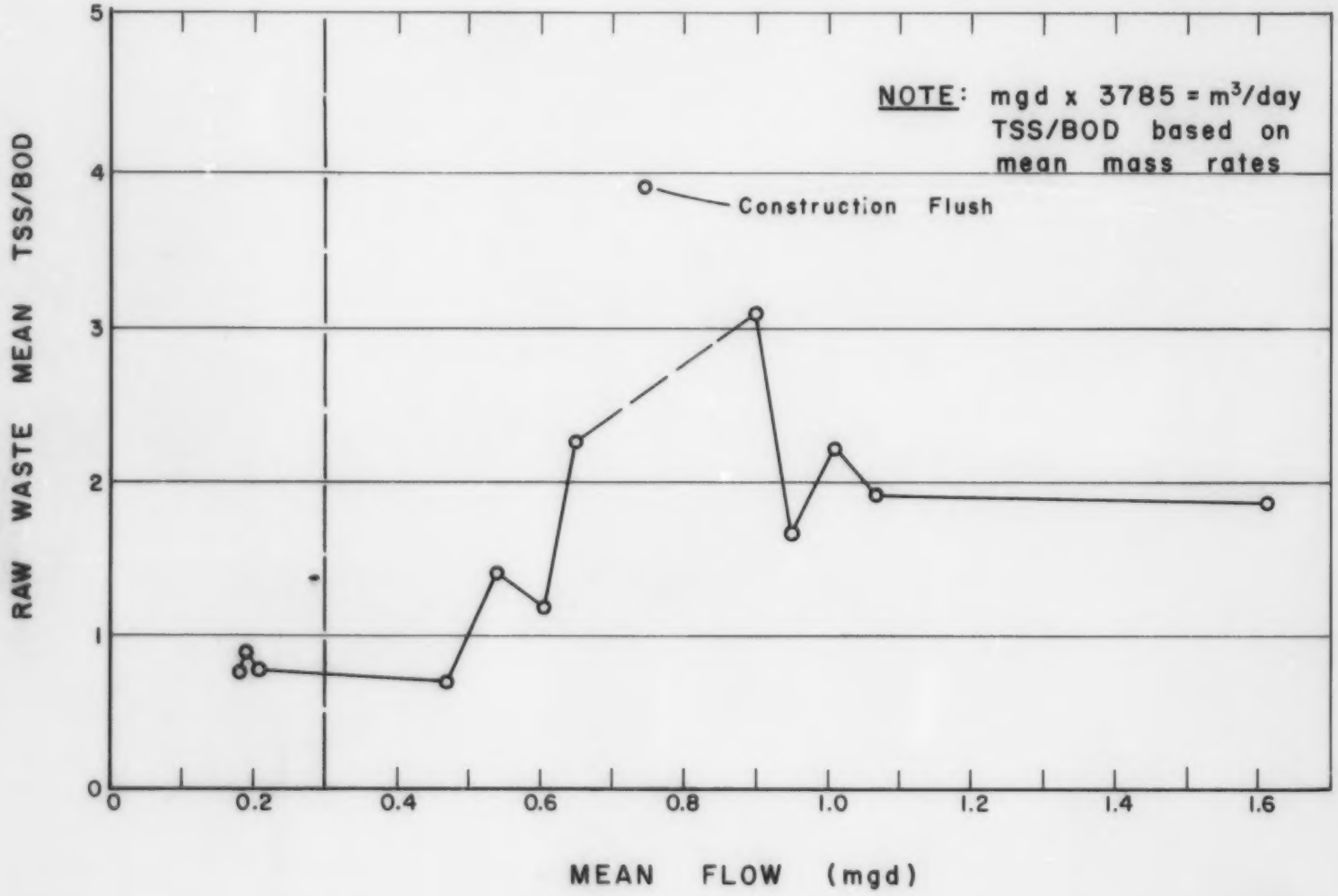

Figure 14: TSS/BOD Ratio vs. Flow

Table 12: Summary of Wastewater Characteristics for Contact Stabilization Treatment of WWF

Process Variable	SURVEY NUMBER 6	9	7	10B	8	11B	10A	11A
Flow (mgd)								
Mean	0.47	0.54	0.61	0.75	0.90	1.02	1.07	1.61
Maximum	0.53	0.65	0.72	0.86	1.22	1.68	1.25	2.31
Input TSS Load (lb/d)								
Mean	536	149	859	988	893	491	498	1136
Maximum	893	-	1074	-	1730	925	-	2890
Input TSS Conc. (mg/l)								
Mean	125	33	166	158	115	55	56	80
Maximum	210	-	226	-	170	73	-	150
Input BOD Load (lb/d)								
Mean	378	162	562	244	273	244	384	781
Maximum	476	-	680	-	572	715	-	1676
Input BOD Conc. (mg/l)								
Mean	88	36	108	39	37	26	43	63
Maximum	112	-	120	-	78	51	-	132
Eff. TSS (mg/l)								
Mean	3	9	2	24	24	18	14	39
Maximum	12	11	3	42	28	44	16	64
Eff. BOD (mg/l)								
Mean	7	11	20	6	>16	7	11	20
Maximum	9	12	27	8	>23	17	14	50

mgd x 3785 = m3/day
lb/d x 0.4536 = kg/day

Table 13: Summary of Operational Characteristics for Contact Stabilization Treatment of WWF

Process Variable	SURVEY NUMBER 6	9	7	10B	8	11B	10A	11A
Contact Time q+r (min.)								
Mean	67	27	53	18	38	16	14	11
Minimum	62	21	47	17	30	10	13	8
Reaeration Time (hr)	1.9	2.9	1.7	1.9	1.6	2.4	1.9	2.2
MLSS (mg/l)								
Initial	4200	3525	3650	4109	5600	3752	5160	3400
Operating	3656	3525	2040	2975	1528	3752	2390	1894
F:M (kg-BOD/day/kg-SS)								
Contact Basin (Operating)	0.50	0.38	1.10	0.70	0.75	0.54	1.35	3.5
Contact + Reaer.	0.35	0.05	0.35	0.08	0.14	-	-	0.29
TASS - Basis	0.21	0.04	0.26	0.04	0.06	0.05	-	0.25
SRS (mg/l)	11700	12900	11700	12550	11400	15800	21300	10480
Aeration Zone	2	1	2	1	2	1	1	1
Reaeration Zone	1	1+2	1	1+2	1	1+2	1+2	1+2

Table 14: Comparison of Contact Stabilization Treatment of WWF

Process Variable	WWF Criterion	SURVEY NUMBER 6	9	7	10B	8	11B	10A	11A
Contact Time (minutes)	>10								
Mean									
Minimum									X
MLSS (mg/l)	>2100								
Initial									
Operating				X		X			X
Reaeration Time (hours)	1-4								
Resp. Rate (mg-0_2/hr/gm-SS)	>2.3								
Contact Basin									
Reaeration Basin									X
Sec. Cl. Ov. Rate (gpd/ft^2)	<1000								
Mean									
Maximum							X		X
Flow (mgd)	-								
Mean		0.47	0.54	0.61	0.75	0.90	1.02	1.07	1.61
Maximum		0.57	0.65	0.72	0.86	1.22	1.68	1.25	2.31
Eff. TSS (mg/l)									
Mean		3	9	2	24	24	18	14	39
Maximum		12	11	3	42	28	44	16	64
Eff. BOD (mg/l)									
Mean		7	11	20	6	>16	7	11	20
Maximum		9	12	27	8	>23	17	14	50
Other Factors					(a)	(b)			

Note: An "X" indicates that plant operation was outside of the range of criteria given.
mgd x 3785 = m3/day gpd/ft^2 x 0.041 = m3/m2/day

(a) TSS load from construction
(b) No primary clarification

effluent BOD concentrations were between 6 and 11 mg/l. Maximum measured effluent TSS concentrations were in the 11-42 mg/l range, while maximum BOD levels were between 8 and 14 mg/l. Peak flows for these same four surveys varied from a low of 0.53 mgd (2006 m3/day) to a high of 1.25 mgd (4731 m3/day).

Figure 15 shows a summary plot of measured effluent TSS and BOD concentrations versus probability of occurrence for all data obtained in the four surveys cited above. Plant flows are also plotted for comparison. From a reliability standpoint this plot shows that for WWF up to 1.25 mgd (4731 m3/day) based on peak flow (1.07 mgd, or 4050 m3/day, based on mean flow) the Clatskanie treatment facility is capable of maintaining effluent quality equivalent to that from well-operated, activated sludge plants.

Reference 27 provides frequency plots such as those shown in Figure 15 for activated sludge treatment under typical variations of DWF. Table 15 summarizes expected effluent TSS and BOD concentrations from these plots and compares them to measured values obtained at Clatskanie under WWF conditions.

It can be seen that under WWF conditions Clatskanie effluent TSS quality is at least equal to that documented in Reference 27, while BOD quality is somewhat better. Reference 27 data are for activated sludge plants without long-term secondary sedimentation (overflow rates greater than 400 gpd/ft2, or 16 m3/m2/day). Most of the Clatskanie data were obtained at overflow rates greater than 400 gpd/ft2 (16 m3/m2/day), except in the lower frequency region (less than about 25%). Secondary clarifier solids loadings at Clatskanie were less than 30 lb/sq ft/day (150 kg/m2/day) in all cases shown.

One other factor relative to these four surveys should be identified: All measured effluent TSS concentrations greater than 24 mg/l were obtained in survey 10B. During this survey primary DAF TSS removal was negligible, while for surveys 6, 9 and 10A removal efficiencies were high (42-65%). Construction activities taking place in the collection system during the survey created a suspended solids load which could not be attributed to stormflow conditions alone, and which was not readily removed in the DAF process. Although influent composite TSS concentrations from the automatic samplers were not particularly high (see Table 10), visual observations made at the plant during this survey indicated a noticeable change in influent turbidity. A supplementary manual grab sample taken at that time yielded a transient concentration of 3780 mg/l.

For the four surveys discussed above, contact basin F:M ratios varied from 0.38-1.35 kg-BOD/day/kg-SS. F:M ratios based on contact-plus-reaeration solids were between 0.05 and 0.35, while inclusion of secondary clarifier solids in F:M calculations lowered this range to 0.04 to 0.21. As indicated in Table 13, one of the four surveys was conducted with

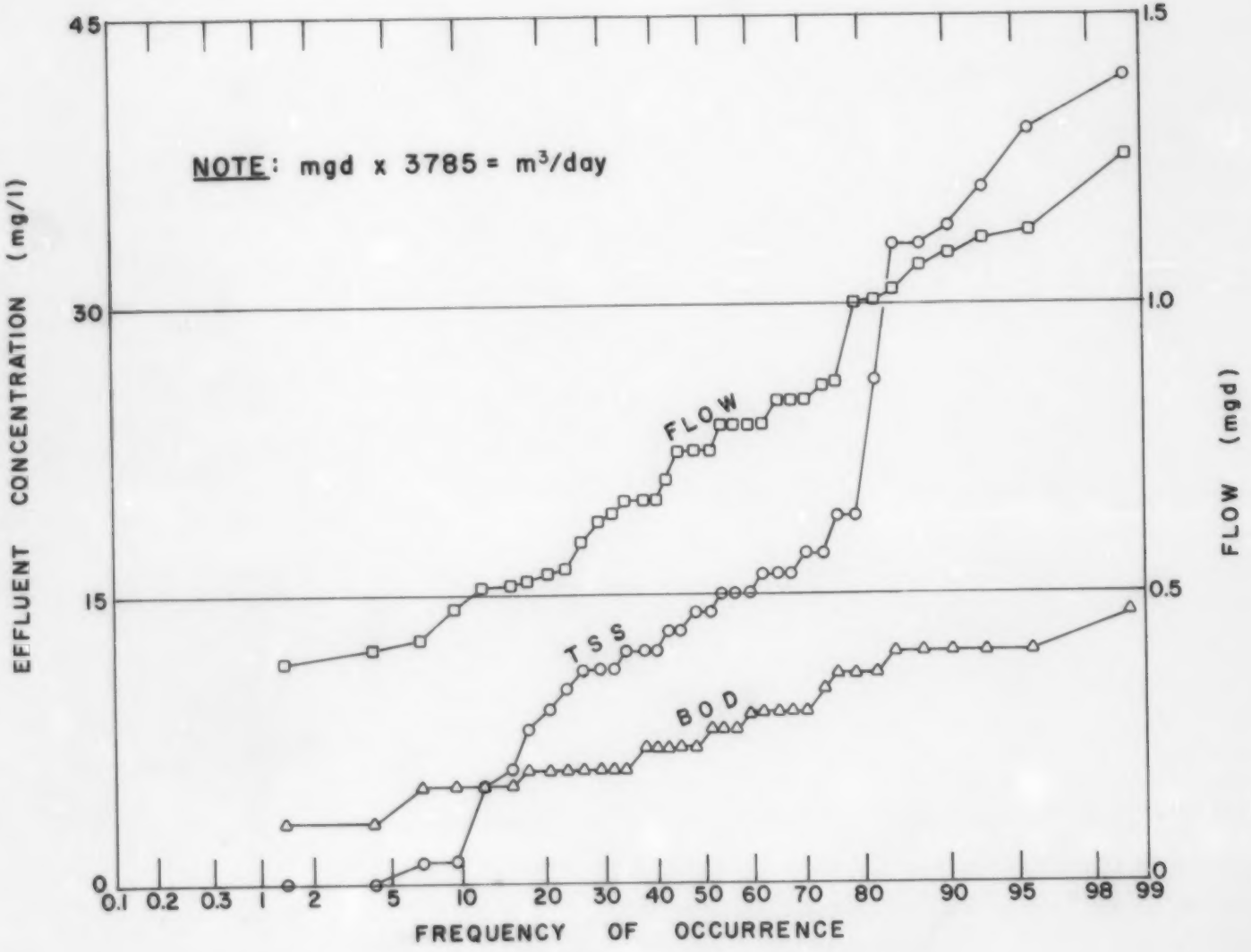

Figure 15: Effluent Quality Frequency Distributions

Table 15: Comparison of Effluent Quality Reliability

Frequency of Occurrence	Effluent TSS, mg/l		Effluent BOD, mg/l	
	Clatskanie	Ref. 27	Clatskanie	Ref. 27
10	1	5	5	6
20	9	8	6	9
30	11	10	7	12
50	14	13	8	17
80	19	18	11	25
85	32	28	-	30
90	34	35*	12	35
95	37	-	-	-
98	41	-	13	-

*Extrapolated beyond 88% frequency to illustrate trend

aeration zone 2 and reaeration zone 1, while the other three were operated with aeration zone 1 and reaeration zone 1+2.

Overall plant performance efficiency and contact stabilization process performance efficiency varied for these four surveys. Mean overall plant removals of TSS were between 84 and 98 percent, while those of BOD were between 81 and 97 percent. Secondary process performance varied from 71 to 98 percent for TSS removal, and from 69-94 percent for BOD removal. Efficiency calculations are affected in part by variations in influent loads (for a constant output load), as well as by functional factors in the process.

A comparison between effluent variability experienced at Clatskanie with contact stabilization/DAF treatment under WWF and that experienced with sedimentation/complete-mix treatment under DWF is given in Table 16. Several observations can be made from the comparison presented.

First, it is apparent that effluent TSS and BOD concentrations discharged at Clatskanie under DWF conditions are somewhat below those indicated in Reference 27 as being representative of activated sludge performance (see Table 15). Secondly, the comparison shows that TSS concentrations discharged at Clatskanie under wet weather flows up to 1.25 mgd (4731 m3/day) are slightly above those discharged with DWF operation. As noted above, TSS values greater than 24 mg/l were associated with a flush from construction activities. Finally, it will be noted that effluent BOD concentrations discharged for DWF conditions are generally above those for WWF operation.

Further inspection of Table 14 shows four surveys in which contact stabilization operation was outside of the range of one or more of the WWF criteria listed. The results from these surveys provide insight into factors which contribute to a deterioration of effluent quality. Three of the latter surveys were within the same flow range, 0.47 - 1.07 mgd (1779 - 4050 m3/day), of those surveys for which operation was within the WWF criteria cited, while one was conducted under WWF conditions which exceeded the plant design capacity of 1.25 mgd (4731 m3/day).

The results from Survey No. 7 are particularly interesting. For this storm mean flow rate was moderate at 0.61 mgd (2309 m3/day) and all WWF criteria were met except for one: The operating level of MLSS was slightly below 2100 mg/l (see Table 13). Although at storm initiation a MLSS concentration of 3650 mg/l was carried in the aeration basin, recycle flow was maintained at 0.18 mgd (681 m3/day) and washout occured lowering the MLSS to an operating level of 2040 mg/l. The MLSS-washout did not affect plant effluent TSS, however, as evidenced by measured concentrations between 1 and 3 mg/l. Secondary clarifier overflow rates for this survey were below 453 gpd/ft2 (18 m3/m2/day) and approximately two process nominal hydraulic detention times (contact and clarifier volumes) based on mean flow were included within the sampling survey.

Table 16: Comparison of Effluent Quality for WWF and DWF Treatment at Clatskanie

Effluent Variable	Regime	Mode	No. of Surveys	Flow(mgd)	Frequency of Occurrence (%) 50	80	90	95	98
					(mg/l)				
TSS	DWF	Sed/CM	3	0.13-0.28	5	10	14	18	24
	WWF	DAF/CS	4	0.38-1.25	14	19	34	36	41
BOD	DWF	Sed/CM	3	0.13-0.28	7	18	20	22	29*
	WWF	DAF/CS	4	0.38-1.25	8	11	12	-	13

mgd x 3785 = m3/day

*Extrapolated

Effluent mean and maximum BOD concentrations (20 and 27 mg/l, respectively) were higher than for the four previously cited surveys which met all WWF criteria. In fact, the entire range of measured effluent BOD concentrations (18-27 mg/l) for survey No. 7 was above the range shown in Figure 15 (3-14 mg/l). Thus BOD removal was apparently affected by the low level of contact MLSS which was maintained in the process, even though a mean contact time of 53 minutes (based on q+r) was available for organic substrate removal. The results from this survey support the previously documented (46) criterion of 2100 mg/l as a lower limit for MLSS in contact stabilization treatment of WWF.

Overall plant removals for survey No. 7 were 99 percent for TSS and 85 percent for BOD. Secondary process TSS and BOD removals were 99 and 82 percent, respectively. Operational characteristics are summarized in Table 13.

Contrast data obtained in survey No. 7 to that obtained in survey No. 11B, for which flows varied from 0.72 to 1.68 mgd (2725-6359 m3/day), and the mean flow was 1.02 mgd (3861 m3/day). In this 11B sampling sequence MLSS concentration was held at a mean operating level of 3752 mg/l with a recycle flow of 0.29 mgd. Even at the high flow rates experienced, in some cases exceeding the design maximum of 1.25 mgd (4731 m3/day), a mean plant effluent BOD of 7 mg/l was observed. The maximum measured effluent BOD concentration was 17 mg/l, and this was associated with the peak flow. Except for the maximum value, all other measured BOD concentrations for this survey (2-13 mg/l) were within the range shown in Figure 15, thus indicating that process BOD removal was generally equivalent to those surveys for which all WWF criteria were met.

The maximum measured effluent TSS concentration for survey 11B was 44 mg/l. This peak was associated with the peak flow of 1.68 mgd (6359 m3/day) resulting in a secondary clarifier overflow rate in excess of 1000 gpd/ft2 (41 m3/m2/day). All other effluent TSS concentrations were in the range of 7-22 mg/l, with clairifier overflow rates between 453 and 856 gpd/ft2 (19-36 m3/m2/day).

Thus, with the exception of peak flow conditions, mean effluent TSS concentration for this survey was 16 mg/l, while that for BOD was 6 mg/l. Mean flowrate, excluding the peak, was 0.96 mgd (3637 m3/day). These mean concentrations, representing treatment capability when process operation is within the range for all WWF criteria, compare favorably to those for similar surveys summarized in Figure 15, and in particular to survey 10A having a similar mean flow (1.07 mgd; 4050 m3/day).

Survey 11A was conducted under WWF conditions which exceeded plant design. Mean flow was 1.61 mgd (6094 m3/day), with a peak of 2.31 mgd (8743 m3/day). Under these survey conditions several process criteria were outside the range for WWF operations, and as a result effluent quality deteriorated. Mean effluent TSS for this survey was 39 mg/l, with transients up to 64 mg/l. Mean effluent BOD was 20 mg/l, with transients up to 50 mg/l.

One survey, No. 8, was conducted in the absence of primary clarification. Mean flow for this survey was 0.90 mgd (3407 m3/day), with a peak value of 1.22 mgd (4618 m3/day). Mean and maximum effluent TSS concentrations for this survey were 24 and 28 mg/l, respectively. Mean and maximum effluent BOD concentrations exceeded 16 and 23 mg/l, respectively*.

For this survey, operating MLSS level was 1528 mg/l; washout of contact basin solids occurred and, as for survey No. 7, this apparently resulted in an increase in effluent BOD values relative to those surveys for which operation was within WWF criteria.

Effluent mean and maximum TSS concentrations for this survey were 24 and 28 mg/l, respectively, which are higher than for survey 10A operating with primary DAF (50% TSS removal) and at a slightly higher flow rate. Operational characteristics are summarized in Table 13.

Primary Clarification

Fifteen surveys conducted during the investigation were designed to study and document primary clarifier performance characteristics (see Table 6). Three were conducted under DWF conditions, as discussed previously in this section (Tables 7-9). The remaining twelve surveys were conducted under WWF conditions using both primary sedimentation and DAF modes of operation. Two experienced operational difficulties such that results were inconclusive. Raw waste characteristics for these surveys have been previously presented in Tables 10 and 11. Primary effluent and process operational characteristics are presented in Appendix B.

Table 17 summarizes performance efficiencies from five surveys using primary DAF and for which optimum operation was sought. TSS removals for these surveys varied between 50 and 66 percent, with mean overflow rates (without recycle) of 570 to 3435 gpd/ft2 (23-141 m3/m2/day). BOD removals were lower (26-45 percent), based on data for the two surveys shown.

Recycle flows, pressures, and aeration rates used during the surveys ranged from 0.14-0.29 mgd (530-1098 m3/day), 40-72 psi (276-496 kN/m2), and 25-50 scfh (0.70-1.40 m3/hr), respectively. Within these ranges performance did not appear to be affected. Overflow rates (means) calculated using raw waste flow plus recycle flow varied from 1115-4236 gpd/ft2 (46-174 m3/m2/day). Mean nominal hydraulic retention times based on raw waste flow were 20-124 minutes, with minima as low as 17 minutes.

Primary sedimentation under DWF was found to yield TSS and BOD removals in the range of 66-73 percent and 39-55 percent, respectively, based on two surveys with mean flows of 0.19-0.21 mgd (719-795 m3/day).

*One third of the samples analyzed yielded residual DO below 1 mg/l after five days incubation.

Table 17: Summary of DAF Performance under Optimum Operation

Regime	Flow (mgd)		Overflow Rate* (gpd/ft2)		Recycle Flow (mgd)	Pressure (psi)	Air Rate (scfh)	Removal Efficiency (%)		Nominal Hydraulic Ret. Time (minutes)*	
	Mean	Max	Mean	Max				TSS	BOD	Mean	Min.
DWF	0.17	0.21	570	669	0.18	68	49	56	-	124	104
WWF	0.54	0.65	1730	2070	0.29	40	50	65	45	41	34
WWF	0.57	0.72	1816	2293	0.17	67	32	61	-	40	31
WWF	0.71	0.85	2269	2707	0.14	72	25	66	-	31	26
WWF	1.07	1.25	3435	3981	0.26	50	35	50	26	20	17

mgd x 3785 = m3/day
gpd/ft2 x 0.041 = m3/m2/day
psi x 6.89 = kN/m2
scfh x 0.028 = m3/hr

*Based on raw waste flow

Overflow rates for these surveys were 605-669 gpd/ft2 (25-27 m3/m2/day).

Under WWF conditions with mean flows in the range of 0.51-0.65 mgd (1930-2460 m3/day) and mean overflow rates up to 2095 gpd/ft2 (86 m3/m2/day), TSS removals by primary sedimentation were in the range of 52-56 percent (two surveys). One survey yielded a BOD removal of 52 percent.

Several factors were found to affect removal efficiencies using the DAF process. These are discussed below.

1. Sludge Withdrawal: One survey was made in which primary settled sludge was not withdrawn prior to conversion to DAF for WWF operation. As noted in Section 4 normal operating procedures require this step prior to start-up of the flotation system.

For this survey, in which mean flow rate and mean overflow rate based on raw waste flow were 0.63 mgd (2385 m3/day) and 2014 gpd/ft2 (83 m3/m2/day), respectively, a TSS removal of only 39 percent resulted. As noted in Table 16, with normal operating procedures removals in the range of 60-65 percent would be expected. Recycle flow, pressure and aeration rate were in the same operational range as those surveys shown in Table 17.

2. Skimmer Speed: High skimmer speed was found to decrease performance drastically. For one survey in which skimmer speed was held at 0.34 rpm, above the normal speed of 0.27 rpm, TSS removal was found to be negligible across the DAF process. Though BOD removal was measurable, it was low at 29 percent. Mean flow and mean overflow rate for this survey were 0.61 mgd (2309 m3/day) and 1969 gpd/ft2 (81 m3/m2/day), respectively. DAF operation for this survey was in the same range as those shown in Table 17.

3. Construction Flush: As noted in the previous section of this chapter, construction activities taking place in the collection system during one of the surveys created a suspended solids load which could not be attributed to storm flow conditions alone. DAF performance during this survey was negligible, even with overflow rates in the range of 2400 gpd/ft2 (99 m3/m2/day). It is uncertain what caused the poor performance observed.

4. High Overflow Rate: In one survey, overflow rates in excess of 5000 gpd/ft2 (205 m3/m2/day), based on raw waste flow, were experienced with nominal hydraulic retention times as low as 10 minutes. Under these conditions it was unable to establish a float and, as a result, performance was negligible.

Standby Sludge Evaluation

The purpose of the standby sludge evaluation was two-fold. First, the condition of the standby biomass needed to be documented since it had

a direct effect on the plant removal efficiency when operating in the contact-stabilization mode. This was shown to be true in the Kenosha Study (46), where it was determined that sludge which was held in the reaeration tank for more than five days produced less satisfactory performance when the plant was switched to the contact-stabilization mode.

The second purpose related to the amount of viable standby sludge which could be maintained. It was felt that the amount of viable standby sludge which could be maintained would have a direct relationship to the size of the "flush" which could be accommodated by the secondary plant in a contact stabilization mode.

The standby sludge mass was developed by channeling all of the waste activated sludge through the reaeration basins. The amount of activated sludge wasted each day was recorded. Primary sludge was also added to the reaeration basins. The purpose of the primary sludge was to provide additional food for the organisms and the amount added varied during different phases of the study. In early phases, the biosolids were maintained in reaeration zone 1 which has a volume of 12,700 gallons (48 m3). In early December, reaeration zone 2 was added to give a total reaeration volume of 28,700 gallons (109 m3).

The methodology for evaluating the standby sludge involved routine analyses using conventional activated sludge monitoring tecnhiques. The analyses were performed on a three times-a-week basis and included; temperature, D.O., MLSS, MLVSS, SVI and Respiration Rate analysis.

The data was plotted using graphical methods and when a more or less steady-state condition was reached, the condition of that sludge was considered to be representative. The standby sludge evaluation was divided into five phases. In most cases the phases culminated with a storm. A week to two week phase duration was required to attain a steady state condition.

The five phases analyzed include: Phase 1 which covered the period from October 1, 1979 to October 17, 1979 and culminated in a small storm on October 18, 1979. Phase 2 which covered the period from November 1, 1979 through November 21, 1979 and ended with a storm on November 22, 1979. Phase 3 which covered the period from December 1, 1979 through December 14, 1979 and ended with a storm on December 14, 1979. Phase 4 which covered the period from January 1, 1980 to January 10, 1980 and ended with a large storm on January 11, 1980. The final phase covered the period from January 18, 1980 through February 12, 1980 and no storm data was available for this phase.

The representative data for each of the phases is shown in Table 18. The table gives the mode in terms of the Reaeration Basins used and the amount of primary sludge added. Also included are sludge viability data and storm performance data. The mean cell retention time (MCRT) is given in terms of time in the reaeration basins and in terms of the total time

Table 18: Standby Sludge Evaluation, Representative Data

Phase	1	2	3	4	5
Period	Oct 1-18, 79	Nov 1-21, 79	Dec 1-14, 79	Jan 1-10, 80	Jan 18-Feb 12, 1980
Reaeration Zone	1	1	1+2	1+2	1+2
Basic(s) Used					
Primary Sludge Added est. (lb/day)	25	0	17	50	50
Sludge Condition					
Standby MCRT days	4.5	4.2	9.5	5	
Total MCRT days	10	8	14	11	
SUI	100	95	95	120	
Temperature, C	14-17	14	13	10	9
% Volitile	75		57	59	
Respiration Rate	10	5	5	3	3
Respiration Rate, 20 degrees C	11	7	7	5	5
Total Sludge Quantity, lbs	950	1050	2500	2400	1300
R. R. x lbs	10,450	7350	17,500	12,000	6500
Storm Performance					
Storm Date	10/18/79	11/22/79	12/14/79	1/11/80	
Peak Flow, mgd	0.53	0.72	0.65	2.30	
Ave Secondary					
BOD Removal (%)	94	82	70	70	
Ave Effluent BOD, mg/l	7	20	13	20	
Ave Effluent TSS, mg/l	3	2	10	39	

in the process and reaeration basins. The repiration rate data is sensitive to temperature, and a second respiration rate is given which is corrected to 20 degrees C. The data was corrected for temperature using the equation:

$$\frac{k_2}{k_1} = \theta^{T_2 - T_1}$$

Where k was the respiration rate,
T1 was 20 degrees C.
T2 was the analyzed temperature
and θ value of 1.047 was used

The respiration rate x total pounds (R.R. x lbs) data represents the respiration rate corrected for 20 degrees C multiplied by the total pounds of standby solids. The respiration rate is a measure of the viability of the sludge and this value times the total quantity of sludge would in theory be directly related to organic load which could be treated in the contact-stabilization mode.

As can be seen in Table 18, the highest R.R. x lbs value was achieved in phase 3. This occurred just after the second reaeration basin was added and the sludge volume was increased considerably. In subsequent phases the respiration rate dropped off and in the final phase raw sewage was added to the sludge in an attempt to decrease the MCRT. This resulted in a thinning of the biomass.

Supplementary Studies

1. Nutrients and Heavy Metals: A series of nutrient and heavy metal analyses were made on composite samples of raw waste, primary effluent and secondary effluent under DWF and storm conditions. The results are summarized in Tables 19 and 20.

A lower concentration of raw waste NH3-N was observed under WWF conditions than under DWF; however, organic nitrogen concentrations were similar. Because of the higher flowrates under storm conditions Kjeldahl nitrogen loads applied to the plant were three times those under DWF conditions. Raw waste total phosphorus concentration under DWF was 5.7 mg/l versus 2.0 mg/l under WWF, yielding loads which were roughly similar.

No nitrogen or phosphorus removal across primary sedimentation was observed under DWF, although some did occur under the stormflow regime. The stormflow composite represents both sedimentation and DAF operation.

Secondary effluent loads of Kjeldahl nitrogen discharged under WWF were about 3.4 times those discharged under DWF, while those for phosphorus were about 1.8 times DWF. Removal of Kjeldahl nitrogen across

Table 19: Comparison of Nutrient Levels in Process Streams

Criterion	DWF*			STORM**		
	Raw Waste	Primary Effluent	Second'y Effluent	Raw Waste	Primary Eff. ***	Second'y Effluent
Flow, mgd	0.21	0.21	0.21	0.91	0.72	0.91
Nitrogen, mg/l						
NH3-N	11.6	10.0	2.5	4.2	4.3	2.0
Org-N	10.4	12.0	4.4	11.0	9.3	3.4
Oxid-N	0.52	0.83	4.9	---	---	---
Kjed-N	22.0	22.0	6.9	15.2	13.6	5.4
Nitrogen, lb/d						
NH3-N	20.3	17.5	12.1	31.9	25.8	15.2
Org-N	18.2	21.0	7.7	83.5	55.8	25.8
Oxid-N	0.9	1.4	8.6	---	---	---
Kjed-N	38.5	38.5	12.1	115.4	81.6	41.0
Total-Phosph.						
mg/l	5.7	5.4	3.6	2.0	1.6	1.5
lb/d	10.0	9.5	6.3	15.2	9.6	11.4

*24-hour composite
**17-hour composite
***Some primary by-passing occured during this event

mgd x 3785 = m3/day
lb/d x 0.4536 = kg/day

Table 20: Comparison of Heavy Metals in Process Streams

Criterion	DWF*			STORM**		
	Raw Waste	Primary Effluent	Second'y Effluent	Raw Waste	Primary Eff.***	Second'y Effluent
Flow, mgd	0.21	0.21	0.21	0.91	0.72	0.91
Metal, mg/l						
Lead	0.025	0.030	---	0.035	0.039	a
Copper	0.044	0.045	0.012	0.037	0.048	a
Zinc	0.14	0.14	0.05	0.26	0.26	0.14
Nickel	0.020	0.018	0.022	0.020	0.030	0.030
Chromium	a	a	a	0.007	a	a
Cadmium	a	a	a	a	a	a
Mercury	b	b	b	b	b	b

*24-hour composite
**17-hour composite
***Some primary by-passing occured during this event
a Less than 0.005 mg/l
b Less than 0.0005 mg/l

mgd x 3785 = m3/day
lb/day x 0.4536 = kg/day

the secondary process dropped from 68 percent for DWF to 50 percent for WWF, based on mass rate loadings. No secondary removal of phosphorus was evident under WWF, though 34 percent was observed under DWF conditions.

Heavy metal concentrations in process streams (Table 20) were varied. Cadmium and mercury were below detectable limits of 0.005 mg/l and 0.0005 mg/l, respectively, for all process streams. The same was true for chromium (Limit = 0.005 mg/l), except for the raw waste under WWF which yielded a concentration of 0.007 mg/l.

No removal of lead, copper, zinc or nickel was observed across the primary process under either DWF or WWF conditions. However, removals did occur in the secondary process. Copper decreased by approximately 87 percent under stormflow conditions and by 73 percent under DWF. Zinc removals were 46 percent for WWF and 64 percent for DWF. Removal of lead under storm conditions equalled 87 percent (DWF results were inconclusive). Nickel was not removed with either WWF or DWF treatment.

Nutrient and heavy metal levels were measured in primary and secondary sludges under DWF conditions. The results are summarized in Tables 21 and 22.

2. BOD/COD Ratios: BOD and COD analyses were conducted on composite samples of raw waste and secondary effluent under DWF and storm conditions. These analyses yielded the following BOD/COD ratios:

Regime	Raw Waste	Secondary Effluent
DWF	0.64	0.35
WWF	0.54	0.14-0.15

The WWF secondary effluent BOD/COD ratio of 0.14-0.15 is roughly half the mean value obtained in the Kenosha study (46). The WWF raw waste ratio of 0.54, on the other hand, is about 1.7 times that obtained at Kenosha, and approaches the value obtained for DWF. These values suggest that there is some difference, not only between the CSO treated at Kenosha and the WWF wastewater treated at Clatskanie, but also in the treated waste discharged. The Clatskanie raw waste ratio is over two standard deviations greater than that at Kenosha; the effluent ratio is roughly equivalent to two standard deviations.

3. Disinfection: Fecal Coliform analyses were performed on secondary effluent (before chlorination) and final effluent (after chlorination). These analyses were performed under different flow conditions with different chlorine contact times. In all cases a 100%

Table 21: Nutrient Levels in DWF Sludges

Nutrient	Nutrient Level (mg/kg-SS)	
	Primary Sludge	Secondary Settled Sludge
NH3-N	900	3259
Org-N	2892	2098
Oxid-N	24	76
Kjed-N	3792	5357
Total Phosph.	1706	13,187

Table 22: Heavy Metal Levels in DWF Sludges

Heavy Metal	Metal Level (mg/kg-SS)	
	Primary Sludge	Secondary Solids*
Lead	20.8	47-101
Copper	47.4	312-403
Zinc	47.4	571-792
Nickel	2.9	15.2-18.2
Chromium	1.9	7.0-8.8
Cadmium	1.0	4.9-6.2
Mercury	0.05	0.36-1.56

*Includes results on mixed liquor solids as well as on secondary settled sludge.

kill rate was achieved. Pertinent data is listed below.

Flow	Contact Time (min.)	C12 Resid (mg/l)	Fecal Coliform Seconday Effl.	Count/100 ml Final Effl.
0.40	96	1.0	10,000	0
0.45	85	1.0	20,000	0
0.45	85	1.0	15,000	0
2.10	18	0.5	10,000	0

4. Volatile Suspended Solids Analysis: Volatile suspended solids analyses were performed on samples for survey numbers 4, 5, 7 10B, 11A, and 11B. The percent volatile data for composite samples of influent, primary and effluent are shown below.

Survey No.	Flow Regime	Mean Flow	Percent Volatile Influent	Primary	Effluent
4	WWF	0.71	92	63	100
5	DWF	0.21	94	90	100
7	WWF	0.61	78	80	63
10B	WWF	0.75	50	46	56
11A	WWF	1.61	63	64	55
11B	WWF	1.02	82	87	74
Average		0.82	76	72	75

As can be seen the percent volatile values varied significantly. It is interesting to note that the highest through plant percentages were obtained in survey 5 which was performed during DWF. The lowest through plant percentages were obtained in survey 10B when the "construction flush" was experienced, and in survey 11A when a significant seasonal flush was experienced.

As discussed earlier, the seasonal flush tended to be more inorganic. There also appears to be a slight trend towards the primary percentages being lower than the influent values. This could suggest that organics are more readily removed in the primary than are inorganics.

Economic Considerations

Capital Cost - - The Clatskanie plant was put on line in September of 1977. The total facility cost including engineering and construction was $1,342,000.

The design dry weather flow for the plant was 0.5 MGD (1900 m3/day) and the design wet weather flow was 1.25 MGD (4700 m3/day). The prinicpal modifications made to accommodate wet weather flows include:

1. A primary clarifier with DAF capability.

2. Reaeration basins for contact stabilization treatment.

3. Additional aeration basin flexibility for DWF and contact stabilization treatment.

4. Additional pumps and meters for contact stabilization treatment.

5. A slightly larger secondary clarifier.

If the cost of these items are subtracted from the facility cost, the resultant would be the cost of a comparable DWF facility. For Clatskanie this cost would be $1,180,000. This compares to a cost of $1,130,000 for similar facilities shown in the EPA publication "Construction costs for Municipal Wastewater Treatment Plants: 1973-1977" (52).

For comparison purposes the annual capital costs were determined. An annual interest rate of 7% was used over a 20 year term. The annual capital cost of the joint dry-wet weather plant is $126,700 per year and the annual capital cost of a comparable dry weather facility would be $111,400.

Operation and Maintenance Costs (O & M) - - The annual cost of running the joint dry-wet facility is $57,200 per year. This covers power, chemicals, maintenance items and two full time operators. Of the $57,200 it is estimated that $1500 is spent for treatment of storm flows. This amount is predominantly labor costs. It covers the time required for routine analysis of the stabilization sludge and the time required to flip the process into and out of the storm treatment mode. Energy costs incurred by treatment of storm flows were estimated to be $50 per year (for approximately 5,000 KWH), which was used to power the pressurization pumps for the DAF unit. No additional power demands could be identified for aeration since a single blower was able to meet demands throughout the wet weather season. The average total energy demand for the plant was 50 KW.

Treatment Cost - - To determine the cost per gallon treatment costs it was first necessary to determine the annual average DWF volume. The present average DWF is 0.18 mgd (680 m3/day), and the year 2000 design flow is projected to be 0.50 mgd (1900 m3/day). This would represent a 5.2 percent annual flow increase and an average annual flow of 114.33 million gallons (mg) (432,740 m3).

In the 1979-1980 wet weather season, the plant was operated in a storm treatment mode for 24 days. In this 24 day period 16.90 mg (63970 m3) were treated. Of this 16.90 mg (6397 m3), 4.32 mg (16010 m3) could be accounted for as DWF. The remaining 12.58 mg (47620 m3) could then be considered the annual storm flow volume.

Dividing the annual costs by the annual flows give the following treatment unit costs:

		Unit Cost
1.	Dry weather flow with comparable dry weather flow plant	$1.46/1000 gal.
2.	Dry weather flow with joint dry-wet plant	$1.50/1000 gal.
3.	Dry weather flow and storm flow with joint dry-wet plant	$1.45/1000 gal.
4.	Storm flow with joint dry-wet plant	$1.33/1000 gal.

1000 gal x 3.79 = m3

Summary - - As can be seen in the preceeding table, the Clatskanie plant is providing treatment for the entire range of flows at approximately the same unit cost as would be expected from a plant designed to treat only the DWF. However, the total annual cost of the joint dry-wet facility is about 10 percent higher than that of a comparable DWF plant.

COST EFFECTIVE ANALYSIS

Infiltration/Inflow conditions and treatment requirements vary greatly from city to city. In order to meet the requirements of Public Law 92-500 and provide for anticipated growth, the circumstances in each city must be considered separately. The purpose of this section is to use information developed in this study to give a specific example of how joint wet/dry weather treatment compared to other possible I/I control and treatment alternatives for the City of Clatskanie, Oregon.

When the system planning was undertaken in 1974 the City of Clatskanie's existing sewage treatment plant had reached its design capacity and a new plant was required to meet more stringent effluent limitations and provide for increasing population. In addition, the system was plagued with large wet weather flows, principally caused by numerious infiltrating collector sewers. The four alternatives considered in this cost effective analysis provide for treatment of the dry weather sewage flow and, in various ways, deal with the infiltration/inflow problem. The four alternatives considered include; joint wet/dry weather treatment, complete sewer system rehabilitation, flow equalization storage, and DWF treatment with WWF bypassing. The DWF treatment with WWF bypassing alternative was included as a basis of comparison for the other three alternatives. The analysis was completed using 1975 construction costs and interest rates.

To further aid in evaluating the alternatives, annual BOD and TSS mass removals were determined for each. To allow determination of the annual mass removals, annual BOD and TSS masses generated by the system were determined. This was accomplished by first expanding the DWF data to an annual rate, and then adding this value to the mass generated by each storm during the wet weather season extending from October, 1979 to February, 1980.

The wet weather mass amount was arbitrarily increased by 15% to compensate for small storms which were not documented, also the mass amount which bypassed the plant during a 5 hour period on January 12, 1980 was estimated and included in the total. The annual BOD and TSS masses generated by the Clatskanie system are as follows:

Component	DWF Annual Mass (lb.)	WWF Annual Mass (lb.)	Bypassed 1-12-80 Mass (lb.)	Total Annual Mass (lb.)
BOD	120,500	3165	35	123,700
TSS	96,000	6340	90	102,430

lb x .4536 = kg

The annual volume of flow for a period beginning February 17, 1979 and ending February 17, 1980 was 103.76 mg (392,732 m3). This gives average annual concentrations for BOD and TSS of 143 mg/l and 118 mg/l respectively.

Alternative I, joint wet/dry weather treatment, was the alternative selected for construction at Clatskanie. This system was described in detail in Chapter 4, and utilizes a dissolved air floatation primary clarifier and a secondary activated sludge process with the capability of contact stabilization treatment in order to handle the high wet weather flows. The plant was sized with a hydraulic capacity somewhat less than the peak flows that are encountered at Clatskanie. During the two year study one storm each year exceeded the hydraulic capacity of the plant. In one case, survey number 3, the entire flow from the system was contained by the plant and reasonable treatment was achieved even though flows did peak at 1.64 mgd (6207 m3/day). In the second storm which exceeded the hydraulic capacity of the plant, survey number 11, a peak flow of 2.31 mgd (8743 m3/day) was recorded at the plant and it is estimated that an additional flow averaging 0.4 (1514 m3/day) was bypassed over a 5 hour period. A peak bypass flow of 0.8 mgd (3028 m3/day) was estimated to occur over a 45 minute period. Treatment at or near the peak flow attained in survey 11 deteriorated and some secondary wash-out did occur.

If the treatment plant were going to be designed and built with the knowledge available today, perhaps the plant would be built larger to accommodate the peak flow, or perhaps the additional capacity would be waived since the bypassing problem only existed one day in the 2 year term of the study. To compensate for the peak flow in the cost effective analysis, a small flow equalization basin was added to the system just upstream of the treatment plant. It was found in the study that the plant performed reasonably well up to a flow of 1.5 mgd. The flow equalization basin was therefore sized to accommodate flows in excess of 1.5 mgd, this required a basin with a capacity of 0.4 million gallons (mg).

The capital cost of the joint wet/dry weather treatment plant was $1,342,000 and the capital cost of a 0.4 mg flow equalization basin would be $54,000 for a total capital cost of $1,396,000. The annual operation and maintenance cost of the plant is $57,200 and it is estimated that an additional $1000 would be required for the basin for a total annual cost of $58,200.

As the population of Clatskanie increases and dry weather flows increase, the available capacity for I/I flows will be decreased. This will amount to approximately 0.3 mgd (1135 m3/day) over the design period. This is not a large flow when considering the peak I/I flow and it is hoped that the City, in their ongoing efforts to eliminate I/I sources, will reduce the peak flow by this amount during the design period.

Plant operating records under DWF and WWF conditions show that an annual mass of 115,180 lb (52,245 kg) of BOD and 94,770 lb (42,987 kg) of TSS can be removed by this alternative. This leaves an annual mass discharged of 8520 lb (3865 kg) of BOD and 7660 lb (3475 kg) of TSS.

Alternative II, complete sewer system rehabilitation would involve line replacements, line sealing, replacement of house sewers, and sealing of manhole covers as defined in the I/I Study performed in September, 1974. It should be noted that a sewer system evaluation survey was not done and no work has been done to positively identify the I/I sources. The sources identified include the lines which were installed in 1910 and a section of line between manhole 1 and manhole 4, which was installed in 1957. All of these need to be replaced, approximately 3400 ft. (1036m). It was further estimated that all lines installed in 1955 and 1957, and 10% of the lines installed after 1957 would have to be chemically grouted, approximately 11,800 ft. (3597 m). It was estimated that 10% of the building services located on 1910 lines, approximately 20, would have to be replaced and there were two manholes which were identified to be raised. Total cost of this work was estimated to be $714,000 and, for the purposes of this analysis, it was assumed that 100 percent of the I/I flow could be elminated. Recent experience on a nation-wide scale has suggested that this may be a bad assumption and perhaps considerably less than 100 percent can be removed. The State of Oregon Department of Environmental Quality has informally suggested that 60% of infiltration and 80% of inflow can be removed.

A new 0.5 mgd DWF treatment plant would have also been required with this alternative. This brings the total capital cost to $1,894,000 and the annual O and M cost would be $55,700. To estimate the annual mass amounts for this alternative, it was assumed that conditions documented during DWF would exist year around.

This would mean that the annual mass amounts generated by the system would be 120,500 lb (54,658 kg) and 96,000 lb (43545 kg) of BOD and TSS respectively.

Anticipating plant removals similar to those experienced under DWF conditions, 112,840 lb (51,183 kg) and 90,160 lb (40,896 kg) could be removed and 7,660 lb (3475 kg) and 5840 lb (2649 kg) of BOD and TSS respectively would be discharged. However, the masses removed would have to include pollutants eliminated with the elimation of I/I. The total amounts removed would then be 116,040 lb (52,635 kg) and 96,590 (43,812 kg) for BOD and TSS respectively.

Alternative III, flow equalization storage, would involve the construction of a storage basin immediately upstream of the treatment plant to catch excessive I/I flows during wet weather. Flow records going back to 1977 when the new plant was completed were analyzed, and it was found that the most severe condition for this alternative occurred in December, 1977. To meet this severe condition, it was determined that a 4.1 mg (15,519 m3) storage basin and a reserve treatment capacity of 0.3 mgd (1136 m3/day) were required. This means that an 0.8 mgd (3028 m3/day) plant would be needed.

The basin could be located along the treatment plant access road on diked bottom land. The basin would cover approximately 4 acres (16,187 m2) and require approximately 1500 ft. (457 m) of dike, an average of 13' high (3.96m). A pump station and automatic control would be required so that flows in excess of the design capacity of the plant would be bypassed to the equalization basin and later, when plant flow subsided, the water in the basin would be pumped back through the plant.

The estimated capital cost of the flow equalization basin was $345,000 and an annual cost of $1,200 would be required for operation and maintenance. The estimated capital cost of a 0.8 mgd (3028 m3/day) was $1,700,000 and the annual operation and maintenance cost was estimated to be $59,000. The total capital and annual costs of this alternative were $2,045,000 and $60,200 respectively. Anticipating treatment similar to that experienced under DWF condition, the annual mass of BOD and TSS removed by this alternative, would be 116,270 lb (52,739 kg) and 96,280 lb (43,672 kg) respectively. The annual mass amounts discharged would be 7,422 lb (3,367 kg) and 6,150 lb (2,790 kg) for BOD and TSS respectively.

An additional implementation problem may exist considering that several homes overlook the proposed location for the equalization basin, and there may have been citizen complaints anticipating an unattractive facility.

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Alternative IV, 0.5 mgd (1893 m3/day) DWF plant and bypassing of WWF, was presented to give a basis with which to compare the other alternatives. The capital cost was estimated to be $1,180,000 and an annual cost of $55,700 was estimated for operation and maintenance. It was anticipated that the plant would achieve normal efficiencies during DWF, but the entire WWF would be bypassed. This would mean that the annual mass removed would be 112,800 lb (51,165 kg) and 90,200 lb (40,914 kg) and the annual mass discharged would be 10,860 lb (4,926 kg) and 12,130 lb (5,547 kg) for BOD and TSS respectively.

Table 23 lists the I/I mitigation and treatment alternatives and their respective costs, annual mass rates and removal efficiencies based on annual mass rates. As can be seen the least expensive of the I/I controlling alternatives is Alternative I, joint treatment, and the alternative with the smallest discharge is Alternative II, sewer system rehabilitation. To evaluate the alternatives further, each was compared to Alternative IV, DWF treatment and WWF bypassing. To facilitate this comparison BOD and TSS values were added together and averaged to give a single pollution unit number. For the same reason BOD and TSS removal percentages were averaged.

Evaluating Alternative I, joint treatment, it was found that the alternative was 14% more expensive than standard DWF treatment as defined by Alternative IV, and its removal efficiencies were 4% higher. An additional measure, the number of dollars spent to subtract one pollution unit from the discharge was $71 for this alternative. The evaluating measures for the three main alternatives are as follows:

Alternative	Total Cost Increase (%)	Removal Efficiency Increase (%)	Dollars Spent Per Pollution Unit Removed
I, Joint Treatment	14	4	71
II, System Rehabilitation	40	5	149
III, Flow Equalization	51	5	201

As can be seen Joint Treatment gives a 4% increase in efficency as opposed to a 5% increase by the other two alternatives, but Joint Treatment costs less. Joint Treatment with the small equalization basin costs 14% more than DWF treatment as opposed to 40% and 51% for the other alternatives and the cost per pollution unit removed was $71 as opposed to $149 and $201 for alternatives II and III.

This comparison is valid for the City of Clatskanie, Oregon. Results of similar analyses may vary significantly for other cities.

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TABLE 23: I/I Mitigation and Treatment Alternatives

Alternatives	Capital Cost ($)	Annual Cost ($)	Total Cost ($)	Annual Mass Removed (lbs.) BOD	Annual Mass Removed (lbs.) TSS	Annual Mass Discharged (lbs.) BOD	Annual Mass Discharged (lbs.) TSS	Removals (%) BOD	Removals (%) TSS
I Joint Dry/Wet Weather Treatment and 0.4 mg Equalization Storage	1,396,000	58,200 p/w 646,600	2,042,600	115,180	94,770	8,520	7,660	93	93
II Sewer System Rehabilitation and 0.5 mgd DWF Treatment	1,894,000	55,700 p/w 618,800	2,512,800	116,040	96,590	7,660	5,840	94	94
III 4.1 mg Equalization Storage and 0.8 mgd DWF Treatment	2,045,000	59,000 p/w 668,800	2,713,800	115,830	96,280	7,860	6,150	94	94
IV 0.5 mgd, DWF Treatment with WWF Bypassed	1,180,000	55,700 p/w 618,800	1,798,800	112,840	90,160	10,860	12,230	91	88

p/w = 20 year Present Worth at 6.625% interest.
Capital and O & M costs are based on 1975 Construction Year.
Annual Mass amounts are based on DWF Baseline studies and WWF documented in the 1979-1980 wet season.

lb. x .4536 = kg, mgd x 3785 = m3/day, mg x 3785 = m3

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Operational Observations

In general terms, the plant was capable of controlling the largest organic loads encountered, provided that the MLSS concentration was more than 2100 mg/l. The pollutants which proved to be difficult to remove were the fine inorganic materials. This is exemplified by the construction flush encountered during survey 10B and the seasonal flush encountered during survey 11A. In both surveys the average effluent TSS was higher than average, 24 mg/l and 39 mg/l respectively and the effluent percent volatile was low- 56 and 55 percent respectively. The solids which appeared to be passing through the plant during these surveys were fine inorganics apparently in the colloidal range. These solids were partially removed by the biological floc in the secondary but the biomass was limited in its capacity for inorganics. These fine inorganics were also difficult to purge from the treatment system after the storm, perhaps with the addition of coagulants these fine particles could be removed by the DAF primary unit. The high flows experienced in survey 11A also contributed to the high TSS concentration in the effluent.

Flip-over, DWF to WWF -- Since process flip-overs had to be done manually, it would have been advantageous to have an operator on duty 24 hours a day. Since 24 hour a day operation was not available during the study, if a storm occurred in the evening, the operator had to return to the plant to flip-over the process and if it occurred in early morning hours the flip-over would not be accomplished until 8:00 a.m. This was an inconvenience to the operators and in certain cases the effluent quality was affected by not flipping-over soon enough.

It would be possible to automate the flip-over to contact stabilization, but it would be difficult to automate the flip-over to DAF. The flip-over to contact-stabilization was accomplished by simply turning off one pump, and starting another. This could be easily automated by simply connecting the pump controls to the plant flow meter. When a pre-set flow was reached the pump controls would be activated, and process would be flipped-over.

If the flip-over to contact-stabilization was automated it would probably be worth the additional expense to automate the RAS recycle rate. The RAS recycle rate could be automated so that the recycle rate would remain at a pre-set percentage of the influent flow. This would help to keep the MLSS concentration in the contact basin above 2100 mg/l.

The flip-over to DAF would be difficult to automate for two reasons. First, it is necessary to drain the sludge off the bottom of the primary unit before the DAF is started. This usually requires operator involvement; and second, it was found that some fine tuning of the DAF unit was required to achieve desirable float conditions for optimum removal.

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APPENDIX A

ANALYTICAL TECHNIQUES NOT LISTED IN STANDARD METHODS

1. Respiration Rate

The oxygen uptake rate (OUR) of the mixed liquor sample was first determined. This involved saturating the sample with oxygen by shaking it in a half-filled bottle. An aliquot of the sample was then poured into a BOD bottle and the drop in DO concentration was measured over a 10 minute period. A D.O. meter was used to measure the concentrations. The OUR was then multiplied by six and recorded as mg/l/hr.

The respiration rate was then calculated by correcting the OUR for the MLSS concentration. To do this the OUR of the sample was divided by the MLSS concentration in mg/l and multiplied by 1000. The units for respiration rate are mg-02/hr/gm-SS.

2. Centrifuge Suspended Solids

The Centrifuge suspended solids analysis was a quick method to determine the approximate MLSS concentration. To perform the analysis mixed liquor was poured into cone shaped, graduated centrifuge tubes. The sample was then centrifuged at 4750 rpm for ten minutes. At the end of the period the solids were compacted at the bottom of the tube. The level of solids in the tube was then read and recorded.

To determine the approximate MLSS concentration it was necessary to run the centrifuge analysis parallel with a gravimetric MLSS analysis. From these parallel analyses, a multiplier was determined which could be used to translate the centrifuge solids level to approximate MLSS. It was necessary to run parallel analyses frequently since changing mixed liquor conditions affected the compactability of the solids.

APPENDIX B

SUPPLEMENTARY DATA

Table B-1: Raw Waste Characteristics for DWF Conditions*

Series	Flow (mgd) Mean	Flow (mgd) Max	BOD Concentration (mg/l) Arith. Mean	BOD Max	BOD Stnd. Dev.	BOD Skewness	TSS Concentration (mg/l) Arith. Mean	TSS Max.	TSS Stnd. Dev.	TSS Skewness
1978	0.18	0.23	186	264	33	0.41	145	267	39	0.71
1979	0.19	0.25	233	387	89	0.52	198	468	118	1.36
No. 5	0.21	0.28	204	320	66	-0.19	157	342	88	0.50
No. 1	0.19	0.25	-	-	-	-	158	198	30	-0.14

Series	Flow (mgd) Mean	Flow (mgd) Max	BOD Mass Rate (lb/day) Arith. Mean	BOD Max	BOD Stnd. Dev.	BOD Skewness	TSS Mass Rate (lb/day) Arith. Mean	TSS Max.	TSS Stnd. Dev.	TSS Skewness
1978	0.18	0.23	289	414	59	-0.10	224	401	60	0.74
1979	0.19	0.25	377	613	149	0.22	328	742	218	1.12
No. 5	0.21	0.28	385	584	165	-0.46	314	799	221	0.61
No. 1	0.19	0.25	-	-	-	-	235	297	52	0.00

*All series were obtained with primary sedimentation and with the complete-mix mode using aeration zones 1+2, except for survey No. 1 which operated with primary DAF

mgd x 3785 - m3/day

lb/day x 0.4536 - kg/day

Table B-2: Primary Effluent Characteristics for DWF Conditions*

Series	Flow (mgd) Mean	Flow (mgd) Max.	BOD Concentration (mg/l) Arith. Mean	BOD Max.	BOD Stnd. Dev.	BOD Skewness	TSS Concentration (mg/l) Arith. Mean	TSS Max.	TSS Stnd. Dev.	TSS Skewness
1978	0.18	0.23	-	-	-	-	-	-	-	-
1979	0.19	0.25	98	168	51	0.12	54	98	24	0.44
No. 5	0.21	0.28	124	162	33	-0.52	56	102	27	0.42
No. 1	0.19	0.25	-	-	-	-	70	115	20	1.12

Series	Flow (mgl) Mean	Flow (mgl) Max.	BOD Mass Rate (lb/day) Arith. Mean	BOD Max.	BOD Stnd. Dev.	BOD Skewness	TSS Mass Rate (lb/day) Arith. Mean	TSS Max.	TSS Stnd. Dev.	TSS Skewness
1978	0.18	0.23	-	-	-	-	-	-	-	-
1979	0.19	0.25	161	270	88	0.04	90	204	54	0.92
No. 5	0.21	0.28	235	365	98	-0.20	107	204	62	0.32
No. 1	0.19	0.25	-	-	-	-	102	144	24	0.08

*All series were obtained with primary sedimentation and with the complete-mix mode using aeration zones 1+2, except for survey No. 1, which operated with primary DAF

mgd x 3785 = m3/day

lb/day x 0.4536 = kg/day

Table B-3 : Secondary Effluent Characteristics for DWF Conditions*

Series	Flow (mgd) Mean	Flow (mgd) Max.	BOD Concentration (mg/l) Arith. Mean	BOD Max.	BOD Stnd. Dev.	BOD Skewness	TSS Concentration (mg/l) Arith. Mean	TSS Max.	TSS Stnd. Dev.	TSS Skewness
1978	0.18	0.23	8	15	3	0.70	6	14	3	0.83
1979	0.19	0.25	11	24	7	1.01	10	17	5	-0.41
No. 5	0.21	0.28	19	26	6	-0.97	15	25	6	0.00
No. 1	0.19	0.25	-	-	-	-	-	-	-	-

Series	Flow (mgd) Mean	Flow (mgd) Max.	BOD Mass Rate (lb/day) Arith. Mean	BOD Max.	BOD Stnd. Dev.	BOD Skewness	TSS Mass Rate (lb/day) Arith. Mean	TSS Max.	TSS Stnd. Dev.	TSS Skewness
1978	0.18	0.23	12	23	5	0.80	9	21	4	0.70
1979	0.19	0.25	19	50	14	1.18	17	32	9	-0.14
No. 5	0.21	0.28	34	52	12	0.06	27	56	14	0.69
No. 1	0.19	0.25	-	-	-	-	-	-	-	-

*All series were obtained with primary sedimentation and with the complete-mix mode using aeration zones 1+2, except for survey No. 1 which operated with primary DAF.

mgd x 3785 = m3/day

lb/day x 0.4536 = kg/day

Table B-4: Summary of Operating Conditions for DNF Series

Operating Conditions	1978 Mean	1978 Maximum	1978 Minimum	1979 Mean	1979 Maximum	1979 Minimum	No.5 (See Table 6) Mean	No.5 (See Table 6) Maximum	No.5 (See Table 6) Minimum
MLSS, mg/l	3930	7133	2510	4690	6180	3650	7560	8250	7535
SRS, mg/l	-	-	-	10025	14400	7200	11800	12000	11500
MLSS, lb	1429	-	-	1144	1510	872	2700	-	-
TASS, lb	-	-	-	1830	2123	1585	-	-	-
Temp., C	17.5	20.0	16.0	15.0	19.0	13.0	14.0	-	-
Resp. Rate, mg-02/hr/gm-SS	13.0	24.0	4.0	11.9	17.0	8.6	5.0	6.0	4.0
SVI	165	303	76	171	207	130	118	129	111
SBL, ft	-	-	-	1.25	2.00	1.00	-	-	-
WAS, lb/day	-	-	-	254	360	180	394	-	-
F:M (MLSS-basis)	-	-	-	0.14	0.29	0.04	0.09	-	-
F:M (TASS-basis)	-	-	-	0.08	0.17	0.03	-	-	-
MCRT (MLSS-basis), days	-	-	-	4.4	5.7	2.5	6.9	-	-
MCRT (TASS-basis), days	-	-	-	7.6	10.7	4.4	-	-	-
Prim. Ov. Rate, gpd/sq ft	575	730	445	605	795	540	670	890	445
Sec'y. Ov. Rate, gpd/sq ft	115	145	90	120	160	110	130	175	90

lb x 0.4536 = kg
ft x 30.48 = cm
gpd/sq ft x 0.41 = m3/m2/day

Table B-5 : Summary of Primary Effluent Concentration Characteristics under WWF*

Survey No.	Mode	Flow (mgd)				BOD Concentration (mg/l)				TSS Concentration (mg/l)			
		Mean	Max	Stnd. Dev.	Skew-ness	Mean	Max	Stnd. Dev.	Skew-ness	Mean	Max.	Stnd. Dev.	Skew-ness
3A	Sed.	0.65	0.84	0.17	-0.18	34	55	19	-0.10	80	151	43	0.74
3B	DAF	1.26	1.30	0.08	-1.07	22	35	12	0.27	102	223	89	0.36
3C	DAF	0.57	0.72	0.09	0.14	-	-	-	-	30	58	14	0.60
3D	Sed.	0.51	0.59	0.05	0.00	-	-	-	-	74	107	27	0.00
3E	DAF	0.63	0.72	0.07	0.15	-	-	-	-	66	81	17	-0.51
4	DAF	0.71	0.85	0.06	1.38	-	-	-	-	48	76	21	0.18
6	DAF	0.51	0.53	0.01	0.75	88	112	16	0.72	125	210	57	0.70
7	DAF	0.61	0.72	0.08	0.00	108	120	7	0.31	166	226	34	0.62
9	DAF	0.54	0.65	0.09	-0.17	36	-	-	-	33	-	-	-
10A	DAF	1.07	1.25	0.07	1.06	43	-	-	-	56	-	-	-
10B	DAF	0.75	0.86	0.07	-0.34	39	-	-	-	158	-	-	-
11A	DAF	1.61	2.31	0.53	-0.68	63	132	29	0.59	80	150	31	0.52
11B	DAF	1.02	1.68	0.29	0.87	26	51	13	0.25	55	73	13	0.00

mgd x 3785 = m3/day

*No primary effluent data for surveys 2 or 8.

Table B-6 : Summary of Primary Effluent Mass Rate Characteristics for WWF*

Survey No.	Mode	BOD Mass Rate (lb/day) Mean	Max.	Stnd. Dev.	Skew-ness	TSS Mass Rate (lb/day) Mean	Max.	Stnd. Dev.	Skew-ness
3A	Sed.	422	748	285	0.02	959	1765	757	0.17
3B	DAF	229	379	137	0.23	1101	2418	978	0.35
3C	DAF	-	-	-	-	142	266	67	0.45
3D	Sed.	-	-	-	-	321	500	127	0.16
3E	DAF	-	-	-	-	355	486	122	-0.11
4	DAF	-	-	-	-	294	539	150	0.43
6	DAF	378	476	66	0.74	536	893	241	0.70
7	DAF	562	681	105	-0.01	859	1074	215	-0.42
9	DAF	162	-	-	-	149	-	-	-
10A	DAF	384	-	-	-	498	-	-	-
10B	DAF	244	-	-	-	988	-	-	-
11A	DAF	781	1676	376	1.05	1136	2890	722	0.85
11B	DAF	244	715	198	1.12	491	925	235	0.48

mgd x 3785 = m3/day
lb/day 0.4536 = kg/day

* No primary effluent data for surveys 2 or 8.

Table B-7: Summary of Secondary Effluent Concentration Characteristics Under WWF

Survey No.	Mode*	Flow (mgd.) Mean	Max.	Stnd. Dev.	Skew-ness	BOD Concentration (mg/l) Mean	Max	Stnd. Dev.	Skew-ness	TSS Concentration (mg/l) Mean	Max	Stnd. Dev.	Skew-ness
2	CM	0.95	1.29	0.21	0.27	16	21	4	0.09	8	16	6	-0.01
3A	CM	0.65	0.84	0.17	-0.18	8	9	1	-0.42	37	52	16	-0.45
3B	CM	1.45	1.64	0.20	-0.67	13	15	2	-0.44	32	56	19	0.14
3C	CM	-	-	-	-	-	-	-	-	-	-	-	-
3D	CM	-	-	-	-	-	-	-	-	-	-	-	-
3E	CM	-	-	-	-	-	-	-	-	-	-	-	-
4	CM	-	-	-	-	-	-	-	-	-	-	-	-
6	CS	0.47	0.53	0.06	-0.52	7	9	2	-0.65	3	12	5	0.97
7	CS	0.61	0.72	0.08	0.00	20	27	3	1.26	2	3	1	-0.13
8	CS	0.90	1.22	0.13	1.14	16	23	8	-0.74	24	28	3	-0.53
9	CS	0.54	0.65	0.09	-0.17	11	12	1	-0.39	9	11	2	-0.46
10A	CS	1.07	1.25	0.07	1.06	11	14	2	0.38	14	16	2	-0.32
10B	CS	0.75	0.86	0.07	-0.34	6	8	1	-0.51	24	42	11	0.34
11A	CS	1.61	2.31	0.53	-0.68	20	50	9	2.06	39	64	10	0.94
11B	CS	1.02	1.68	0.29	0.87	7	17	5	0.91	18	44	10	1.26

*CM=Complete-mix; CS=Contact Stabilization

lb/day x 0.4536 = kg/day

Table B-8 : Summary of Secondary Effluent Mass Rate Characteristics for WWF

Survey No.	Mode *	BOD Mass Rate (lb/day) Mean	Max	Stnd. Dev.	Skew-ness	TSS Mass Rate (lb/day) Mean	Max	Stnd. Dev.	Skew-ness
2	CM	127	205	52	0.21	62	129	44	0.13
3A	CM	43	54	8	0.30	189	312	80	0.36
3B	CM	156	188	43	-0.66	387	70	239	0.24
3C	CM	-	-	-	-	-	-	-	-
3D	CM	-	-	-	-	-	-	-	-
3E	CM	-	-	-	-	-	-	-	-
4	CM	-	-	-	-	-	-	-	-
6	CS	27	38	9	-0.60	11	40	15	0.99
7	CS	103	128	17	0.24	10	18	5	0.12
8	CS	124	226	70	-0.22	184	265	45	0.48
9	CS	49	65	11	-0.27	42	60	14	-0.27
10A	CS	99	126	14	0.49	130	146	16	-0.53
10B	CS	37	53	10	-0.49	153	291	75	0.40
11A	CS	239	366	57	1.01	548	1233	292	0.75
11B	CS	67	238	66	1.43	173	616	154	1.87

lb/day x 0.4536 = kg/day

*CM=Complete-mix; CS=Contact Stabilization

Table B-9: Summary of Primary Process Characteristics for WWF

Process Characteristics	Survey Number 3A	3B	3C	3D	3E	4	6	7	9	10A	10B	11A	11B
Mode	Sed.	DAF	DAF	Sed.	DAF	DAF	DAF	DAF	DAF	DAF	DAF	DAF	DAF
Recycle Flow (mgd)	-	0.22	0.17	-	0.22	0.14	0.22	0.18	0.29	0.26	0.26	0.22	0.20
Pressure (psi)	-	50	67	-	60	72	60	68	40	50	50	60	62
Air Rate (scfh)	-	85	32	-	35	25	45	55	50	35	35	15	30
Temperature (°C)	-	10.0	10.0	-	10.0	11.0	16.0	13.0	12.0	12.0	12.0	9.5	9.0
Ov. Rate, Q Basis (gpd/ft2)													
Mean	2095	4025	1815	1644	2014	2269	1640	1969	1730	3436	2412	5132	3251
Maximum	2675	4140	2293	1879	2293	2707	1688	2293	2070	3981	2739	7353	5350
Stnd. Dev.	552	256	316	173	229	193	32	270	298	252	241	1714	951
Skewness	0.18	1.07	0.14	0.00	0.15	1.39	0.75	0.00	0.17	1.06	0.34	0.68	0.87
Ov. Rate, Q+R Basis (gpd/ft2)	-	5318	2357	-	2707	2707	2197	2516	2643	4204	3217	5828	3885
Hyd. Ret. Time, Q Basis (hr)													
Mean	0.59	0.29	0.66	0.71	0.58	0.51	0.73	0.61	0.69	0.34	0.48	0.27	0.38
Minimum	0.44	0.28	0.51	0.62	0.51	0.43	0.71	0.52	0.56	0.29	0.43	0.16	0.22
Stnd. Dev.	0.16	0.01	0.11	0.07	0.06	0.03	0.01	0.08	0.12	0.02	0.04	0.15	0.09
Skewness	0.30	1.07	0.24	0.13	-0.05	-1.28	0.75	0.05	0.40	1.10	0.63	1.95	0.29

mgd x 3785 = m3/day
gpd/ft2 x 0.041 = m3/m2/day
psi x 6.89 = kN/m2
scfh x 0.028 = m3/hr

Table B-10: Summary of Secondary Process Characteristics for WWF

Process Characteristics	Survey Numbers													
	2	3A	3B	3C	3D	3E	4	6	8	9	10A	10B	11A	11B
Aeration Time, Q Basis (hr)														
Mean	1.12	1.66	0.71	-	-	-	1.45	1.51	0.80	0.64	0.31	0.46	0.25	0.35
Minimum	0.80	1.22	0.63	-	-	-	1.21	1.33	0.60	0.53	0.27	0.40	0.15	0.20
Stnd. Deviation	0.24	0.47	0.11	-	-	-	0.10	0.22	0.11	0.12	0.02	0.05	0.14	0.09
Skewness	0.13	0.30	0.84	-	-	-	1.29	0.55	0.69	0.42	0.84	0.32	1.98	0.31
Aeration Time, Q+R Basis (hr)														
Mean	-	-	-	-	-	-	-	1.12	0.64	0.45	0.23	0.30	0.19	0.27
Minimum	-	-	-	-	-	-	-	1.03	0.50	0.35	0.21	0.28	0.13	0.17
Stnd. Deviation	-	-	-	-	-	-	-	0.09	0.07	0.07	0.01	0.02	0.08	0.05
Skewness	-	-	-	-	-	-	-	0.44	0.75	0.09	1.01	0.68	1.58	0.42
Clar. Ov. Rate (gpd/ft2)*														
Mean	599	414	917	-	-	-	448	319	566	342	678	477	1014	642
Maximum	811	528	1031	-	-	-	535	333	767	409	786	541	1453	1057
Stnd. Deviation	136	109	130	-	-	-	38	10	86	59	50	48	339	188
Skewness	0.27	-0.18	0.67	-	-	-	1.39	0.33	1.14	0.17	1.05	0.34	0.68	0.87
Cl. Hyd. Ret. Time (hr)*														
Mean	3.76	5.53	2.39	-	-	-	4.83	7.76	4.10	6.81	3.36	4.82	2.69	3.78
Minimum	2.66	4.08	2.09	-	-	-	4.03	6.80	3.00	5.55	2.89	4.21	1.57	2.15
Stnd. Deviation	0.83	1.58	0.39	-	-	-	0.36	1.16	0.52	1.24	0.22	0.50	1.54	0.93
Skewness	0.11	0.30	0.83	-	-	-	-1.28	0.55	0.82	0.41	0.88	0.58	1.98	0.28

*Based on raw waste flow; gpd/ft2 x 0.041 = m3/m2/day

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

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16. ABSTRACT

This report describes the two year plant scale evaluation of physical and biological processes used for joint treatment of dry weather and storm generated sanitary sewer flow. The project was conducted in Clatskanie, OR at the City's new joint dry/wet weather sewage treatment plant. The purpose of the project was to research and define the performance capability of the new plant for potential nation-wide application. As part of the project, economic and technical criteria for developing alternative I/I control methodologies were reviewed and are presented as part of the report.

Joint dry/wet weather treatment, when compared to other I/I control measures at Clatskanie, was found to be nearly as effective and the cost was significantly less. The plant is a standard activated sludge plant except that the secondary can be switched to a contact stabilization mode and the primary clarifier can be run in a dissolved air flotation (DAF) mode. The present dry weather flow (DWF) is 0.19 mgd (719 m^3/day) and with DAF-contact stabilization operation wet weather flows (WWF) more than 6 times DWF can be effectively treated. Based on annual mass rates, BOD and TSS removals were 94% under DWF conditions and during WWF, with peak flows ranging from 3 to 12 times DWF, BOD and TSS removals were 73% and 71% respectively.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Sewage Treatment, Infiltration, Inflow, Overflows-sewers, Wastewater, Sewage, Waste Treatment	Dissolved Air Foltation, Contact Stabilization, Infiltration/Inflow Control	

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